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TECHNOLOGY TRANSFER OF FIBER REINFORCED POLYMER (FRP) COMPOSITES USED FOR BRIDGE REHABILITATION AND RETROFIT

(DRAFT REPORT)

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16. Abstract

Since the early 1990's there has been a great deal of attention paid to the nation's declining infrastructure. Both Federal and State agencies have been under increasing pressure to repair their infrastructure using whatever means were available.

Also during this time frame Universities and other research institutes began research on the use of Fiber Reinforced Plastics (FRP) for the repair and seismic upgrade of bridges. Support for much of this research was initiated by DARPA. Using Defense conversion money research was initiated to find uses for strategic materials such as, but not limited to carbon fibers.

Early expectations were that these materials would offer another tool to the construction industry and also give owners an economical solution to their bridge problems. One of the first agencies to take the step of transferring the technology from the research and development phase to the demonstration phase was The California Department of Transportation (Caltrans).

During the same period, Alliant Techsystems, and Hexcel Corporation of Salt Lake City, Utah were becoming more involved with the research and development for this application at the University of California, San Diego, through DARPA. Alliant Techsystems, being a Utah company involved in designing FRP structures and Hexcel Corporation, manufacturer of Carbon and also located in Utah approached the Utah Department of Transportation (UDOT) with the intention of transferring this technology from the research and development phase into a demonstration phase and hopefully later, a fully qualified commercial solution to bridge problems.

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2. Acknowledgement

Acknowledgement of all the parties involved is difficult since there were so many agencies and individuals who contributed significantly to this program.

The Utah Department of Transportation deserves special acknowledgement. From the Research & Development Group with their vision to introduce a new experimental technology to the State, and the Structural Engineering Group who embraced the technology and learned to adapt to the special engineering calculations necessary to insure a safe and economical installation. The management of UDOT also is acknowledged for the support and vision for this new technology.

Individually within UDOT there are three people who deserve special acknowledgement for their belief in and work towards this successful technology transfer. Dal Hawks was in charge of the initial phase of the program. Working with university and industrial partners he was instrumental in the introduction of this technology to UDOT. Mr. Sam Musser contributed throughout the program and kept the vision alive. Last, Mr. Doug Anderson worked in the background and helped guide the program.

Dr. Larry Reaveley of the University of Utah was involved throughout the program but in its early stages promoted the concept of FRP repair to UDOT to extend the life of existing structures.

The Federal Highway Administration was instrumental in the program with the funding provided through out the development and implementation of the technology and ultimately, the successful technology transfer.

Special acknowledgement is given to Ralph Nuismer, Ph.D. formerly of Alliant Techsystems for his vision and support in the early stages of the program and also to Mr. Fred Policelli for his vast knowledge of FRP systems and his assistance in implementing these materials into the construction environment.

All the consultants, engineering services, material suppliers, students, and construction partners are acknowledged for their tireless effort to make this program a success. The willingness and opened mindedness shown by all is the major reason for the success of the program.

We wish also to acknowledge The Utah State Prison for providing labor on the Highland Ave. project and the I-15 project.

Last the authors wish to thank everyone individually for their contribution to the success of this program. Each and every person in the program bought into the technology and did everything necessary to make it a complete success.

1. INTRODUCTION

Since the early 1990's there has been a great deal of attention paid to the nation's declining infrastructure. Both Federal and State agencies have been under increasing pressure to repair their infrastructure using whatever means were available.

Also during this time frame Universities and other research institutes began research on the use of Fiber Reinforced Plastics (FRP) for the repair and seismic upgrade of bridges. Support for much of this research was initiated by DARPA. Using Defense conversion money research was initiated to find uses for strategic materials such as, but not limited to carbon fibers.

Early expectations were that these materials would offer another tool to the construction industry and also give owners an economical solution to their bridge problems. One of the first agencies to take the step of transferring the technology from the research and development phase to the demonstration phase was The California Department of Transportation (Caltrans).

During the same period, Alliant Techsystems, and Hexcel Corporation of Salt Lake City, Utah were becoming more involved with the research and development for this application at the University of California, San Diego, through DARPA. Alliant Techsystems, being a Utah company involved in designing FRP structures and Hexcel Corporation, manufacturer of Carbon and also located in Utah approached the Utah Department of Transportation (UDOT) with the intention of transferring this technology from the research and development phase into a demonstration phase and hopefully later, a fully qualified commercial solution to bridge problems.

The discussions between Alliant Techsystems, Hexcel Corporation and UDOT resulted in a program to study the application of FRP materials to Utah bridges. In addition, other industrial partners were involved at this time period including Akzo Nobel, Fabric Development, Inc., Thiokol, XXsys Technologies, Inc., and Zoltek. Universities in Utah, namely the University of Utah and Utah State University were brought into the team to work on solutions involving FRP materials. The Federal Highway Administration through the Priority Technologies Program funded the early research.

This report embodies the work done by UDOT, its consultants, and its industrial and construction partners to transfer this technology from a research and development phase into a fully commercialized solution for bridge infrastructure repair. The present report covers the history of all research and demonstration projects that UDOT sponsored, culminating with a detailed report on the seismic retrofit of the State Street Bridge under Interstate 80.

Special provisions (specifications) were developed for the carbon FRP composite column jackets of several bridges on Interstate 80 in Salt Lake City, Utah; in addition, special provisions were developed for a carbon FRP composite bridge bent wrap for the seismic retrofit of the State Street Bridge, at Interstate 80 in Salt Lake City, Utah. The specifications included provisions for materials, constructed thickness based on strength capacity, and an environmental durability reduction factor. Surface preparation, finish coat requirements, quality assurance provisions, which included sampling and testing, and constructability issues regarding the application of fiber composite materials in the retrofit of concrete bridges are also described.

This report is also intended to educate the reader on how a technology transfer program can be successfully implemented. UDOT's progressive attitude coupled with it's research and construction partners as well as the Federal Highway Administration have taken a technology initially developed for the aerospace and sporting goods industry and helped make it a viable alternative to conventional methods of repair and retrofit of concrete structures.

Spin off technologies resulting from this program is also discussed, as well as the university degrees that were supported by this program.

3. History

The history section is intended to give the reader a picture of how the implantation of FRP by UDOT evolved through two major projects and culminated in the construction contract for the seismic upgrade of the State Street Bridge under I-80.

Detailed reports summarizing all technical aspects of these projects were written and submitted to UDOT. It is not the intent of the history section to cover these individual projects in detail, but to put them in context as the results apply to the seismic retrofit of the State Street Bridge under I-80.

Appendix C is a graphical representation showing a time line of the FRP programs in Utah.

Highland Drive

In May 1995 UDOT's Structures and Research Divisions, Hercules, Inc., The University of Utah and Utah State University submitted a research proposal to the FHWA under the Priority Technologies Program.

The focus of this research proposal was to build a replica of the bridge bent at Highland Drive and I-80, reinforce the replica with FRP materials, test the replica and using the technology tested, reinforce the actual Highland Drive and I-80 bridge.

Hercules intended to perform the reinforcing task but because of internal business decisions (mainly not to enter into the contracting business) they contacted XXsys Technologies, Inc. in San Diego. XXsys Technologies was a start-up company working closely with The University of California San Diego (UCSD), Caltrans, Hercules, Inc. and Ciba Geigy Corp. on a DARPA defense conversion contract to commercialize the use of carbon fibers (a strategic material) in infrastructure industry.

XXsys Technologies Inc. was invited to visit Utah to discuss the project and ascertain weather the project could be done using XXsys patented technology. After the visit XXsys determined that the project was possible using their technology. At this time Ciba Geigy Corp. dropped out of the consortium announcing that they were unable to supply the raw material for this project.

The reason for the discussion above is to put into perspective how the final material supplier was chosen for this project. While conducting research at UCSD XXsys Technologies was in the process of evaluating materials supplied by TCR Composites, Ogden, Utah (a division of Thiokol Corp.) for use in the infrastructure market. After a joint meeting with UDOT, University of Utah, Utah State University, Hercules, XXsys Technologies and TCR Composites, it was decided that the materials used would be TCR Composites TCR Epoxy # UF3325-95 resin system impregnated into Akzo fibers for the columns and Zoltek fibers woven into a fabric by Fabric Development Corporation for the bent cap. The afore mentioned products were tested and in use at TCR Composites and had a successful history.

XXsys intended to use the TCR Composites' system for retrofitting the columns using its ROBO Wrapper® technology of filament winding the composite material onto the columns and curing the material at elevated temperatures and an impregnated fabric material supplied by TCR Composites on the cap beam, also being cured at elevated temperature.

The samples at Utah State University were successfully wrapped the last week of August 1996. The next week, September 1, 1996 though September 7, 1997, the Highland Ave. Bridge was also successfully wrapped.

University of Utah Slab

During 1996 The University of Utah embarked on two additional testing programs, one involving the use of FRP materials in straightening corroded bridge deck slabs and also reinforcing and testing T-Joints.

University Partners

The University of Utah was responsible for the design and analysis of the FRP on both the test samples and the Highland Ave. Bridge.

Utah State University was responsible for building the full-scale replica of the bridge and conducting all the testing. The test was done using quasi-static techniques.

Industrial Partners

The industrial partners for this project were:

XXsys Technologies, Inc. supplied the Robo Wrapper® machine, curing ovens and labor to apply the FRP materials to both the Utah State University test elements and the Highland Ave. Bridge.

Hercules Corp. was instrumental in initiating the project and also served as an oversight-engineering firm giving advice on the design and installation.

Using fibers supplied by Akzo Fibers of Knoxville, TN TCR Composites supplied the resin (TCR Epoxy # UF3325-95) and labor to impregnate the fibers and deliver them in a proper form suitable for use with the Robo Wrapper®. The fabric material provided to the project by Fabric Development using Zoltek fibers was also impregnated with the above-mentioned resin.

Akzo Fibers provided all the carbon fiber for column wrapping. This material was supplied as a 50K tow.

Zoltek provided the 48 K fibers to the project. This material was sent to Fabric Development who wove the material into the form necessary to apply to the bent cap.

Fabric Development is a weaving company that wove the 48K tows into the fabric for the bent cap.

The industry partners worked very well with the owner, university partners and other industry partners. All of the companies involved had technical representatives on site during various phases of the installations and testing. In particular, TCR Composites provided daily assistance during the entire project.

I-15 Research

During the time period the research work was proceeding on the use of FRP materials in Utah, the Utah Department of Transportation (UDOT) awarded a design/build contract for rebuilding the I-15 corridor. Wasach Constructors was the consortium awarded the contract.

Demolition of existing I-15 bridges was scheduled. Working closely with UDOT and Wasach Constructors the test bed bridge bents were chosen. The South Temple bridges were excellent candidates for the proposed research program since these structures exhibited many if not all of the defects that exist on bridges in Utah and in other state around the nation.

The teamwork shown on this project was exceptional considering the test bridges were in the critical path of Wasach Constructors project.

University Partners

The lead University for the project was the University of Utah, responsible for the design of the FRP application, testing and overall coordination. The University of Utah utilized many graduate students for various tasks on the project including the actual installation of the FRP materials.

As a side note, the experience gained by the students from the University of Utah and Utah State University was invaluable. It is not very often that a group of students get to work on a “real world” project. They were shown how a project moves from the laboratory to a commercial project.
Utah State University provided the technology and the students to conduct vibration tests on the structure.

As mentioned above both the north bound and south bound bridges at South Temple were retrofit and tested. The north bound section was done first, the results of that test program aided in calibrating analytical models and outline procedures for design of cap beam-column joints and cap beams and column retrofit on the north bound bridge and later on the I-80 Interstate, State Street Bridge contract.

Industrial Partners

Wasach Constructors were the general contractor for the I-15 design build contract.

Penhall provided labor and demolition of the structures after the test.

Sika Corporation provided the materials used on the project.

Hydortech was the contractor who was responsible for the application of the FRP materials on the project. Hydortech used many of the University of Utah Students to assist in this effort.

Navlight Composites was contracted to oversee the installation, assist with quality issues as they arose on the project and provide a final report on the project.

As with the Highland Ave. project the contractors, universities, material suppliers and UDOT worked very well together realizing that the success or failure of the tests would rest on performing all the work and tests as part of the overall I-15 design build contract.

Project Scope

The project scope details for the southbound bridge are covered below. It would be redundant to give the full details of both phases of the projects. As stated above, the southbound bridge design was a result of initial testing on the northbound bridge.

Introduction

This project centered on installation of FRP materials on I-15, Bridge No. 58, Bent No. 6 that has corroded due to environment factors such as freeze-thaw and salt applications over the years.

This was a unique opportunity to conduct a full-scale field test on an existing structure using carbon composite technology. The I-15 corridor reconstruction allowed the team to retrofit and test an existing bridge prior to its demolition.

This project is a continuation of the project conducted on Bent 5 and Bent 6 of the northbound I-15 at the same location. The results of that test were used to fine-tune the design for this project.

The materials used on the project consisted of the carbon fabric, adhesive and a resin system supplied by Sika Corporation.

The primary object of this program was to determine the ability of composite materials to restore the original strength of the structure. Moreover, the proposed test program aided in calibrating analytical models and outline procedures for design of cap beam-column joints and cap beams and columns.

The mechanical test consisted of installation of a full-scale actuator to perform quasi-static tests. The tests will be carried out on Bent No. 6. These tests were conducted to destruction of the structure.

Other tests conducted on the structure during the duration of the project included Geo-Pier tests conducted by the University of Utah and vibration tests conducted by Utah State University.

Bridge No. 58 Bent No. 6

South Bound I-15 @ South Temple

Sika Corporation retrofitted the southbound I-15 Interstate Bridge located in Salt Lake City, Utah. The bridge is located at the South Temple Street Overpass on Interstate 15. The bridge has 8 bents and 9 spans. The tests were conducted on Bent No. 6. The reason for choosing this bent is the convenient location, especially the distance from the railroad tracks.

Sika Corporation retrofitted 3 square columns and cap beam on Bent No. 6. The columns were 3’ x 3’ x 24’ (0.91m x 0.91m x 7.32m) tall. The cap beam cross-section was 36” x 48” (0.91m x 1.22m). The cap was 64.5 feet long (19.67m). This report starts with details of initial condition of the bridge. The detailed design, materials used, cost of materials and procedure are included later in this report. Condition of the Bent
The bent had large cracks and holes in the concrete column in the areas to be retrofitted. Concrete had spalled off leaving holes approximately 1’ x 4’ (0.31m x 1.22m) at the corners. Rebar was exposed on the cap beam. Shot Crete was applied to restore the cap to its original dimensions after all loose concrete was removed with water blast.

All of the corners had to be rounded during the retrofit. Shot Crete on the I-beams had to be rounded off. Column surfaces had to be cleaned prior to any material being applied. This step can be speeded up if the surface is water washed prior to the retrofit work.

Cap Beam Showing Exposed Rebar and Spalled Concrete

Cap Beam Repaired with Shot Crete

Repaired and Cleaned Column

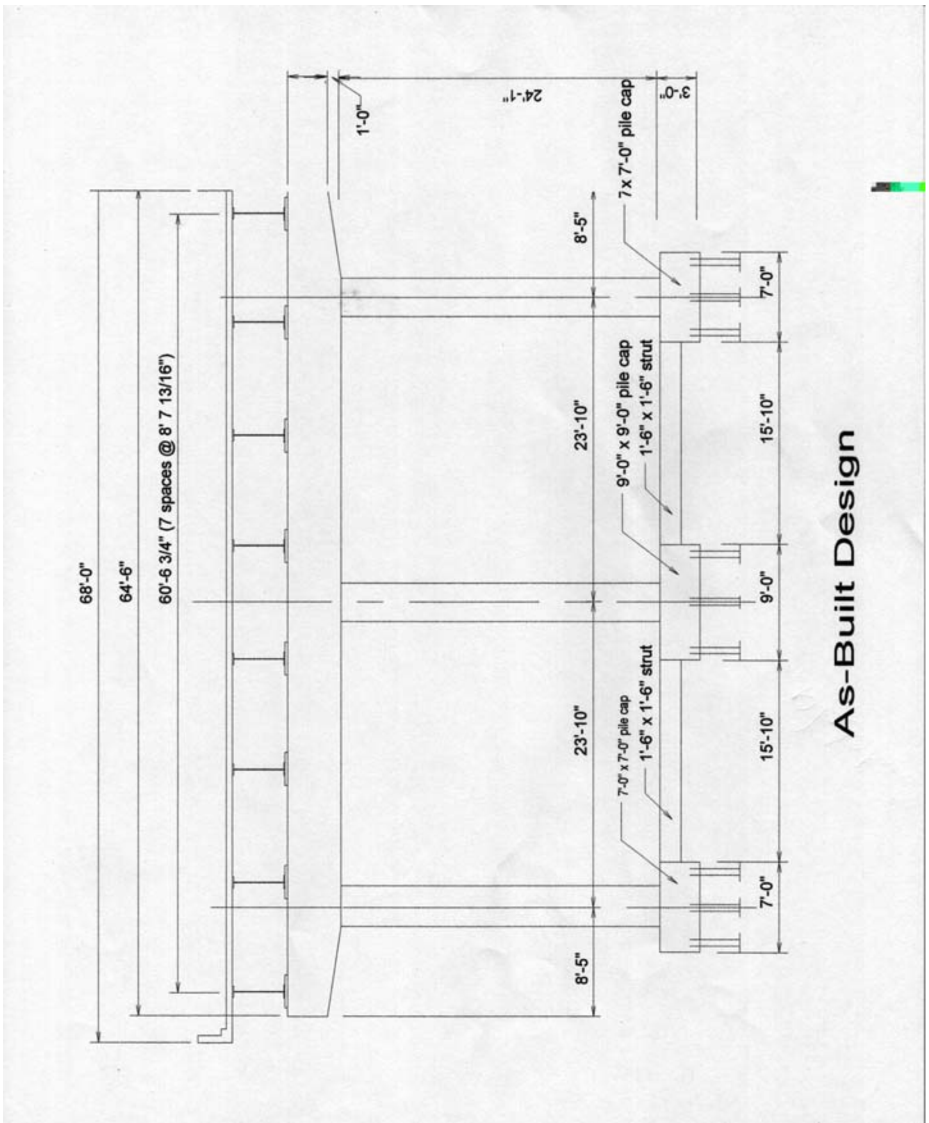
Design

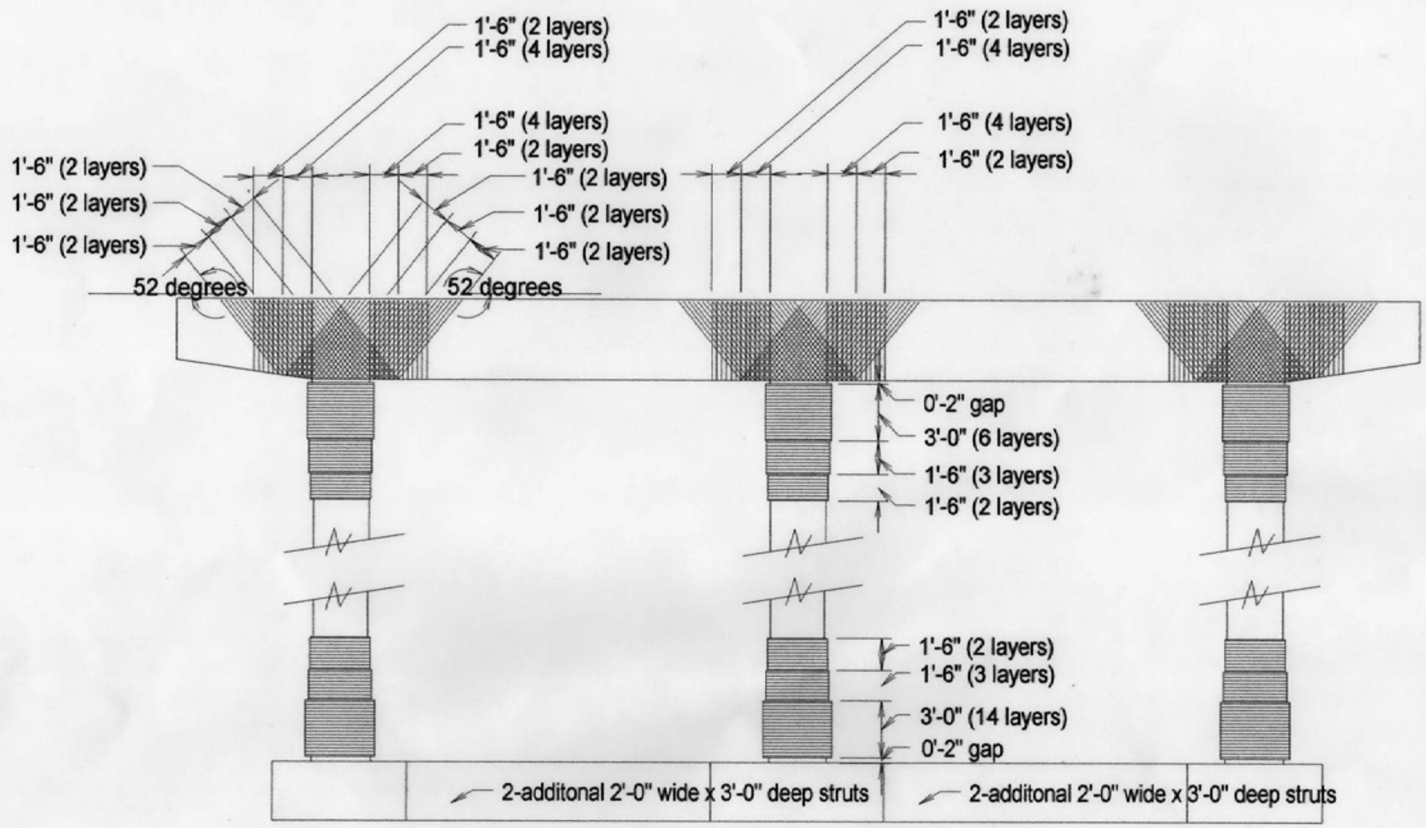
The following figures show the overall dimensions of the retrofit design for Bent #6. The design shows three separate wraps per column, i.e. bottom, top, and column-cap joint. The bottom and top sections are mainly composed of 0-degree wraps. The cap contains of +52, -52 and 90 degree wraps.

The design is based on 18” wide carbon fabric. In order to achieve 3-foot wide wraps, two 18” wide fabrics are placed side-by-side. All the fabric for the cap beam use 18” wide fabric. The vertical straps use 6” wide fabric. The 0-degree wraps in the cap beam hold the 52-degree joint wraps in place through a clamping mechanism. The 6” straps hold the joint together by relieving the vertical column bars of the high tensile forces, and by providing a positive connection between the cap beam joint and the column.

The design shows girders present along the cap beam. The girder positions turned out to be incorrect in the field. As a result minor changes were made to accommodate this (included in Procedure section).

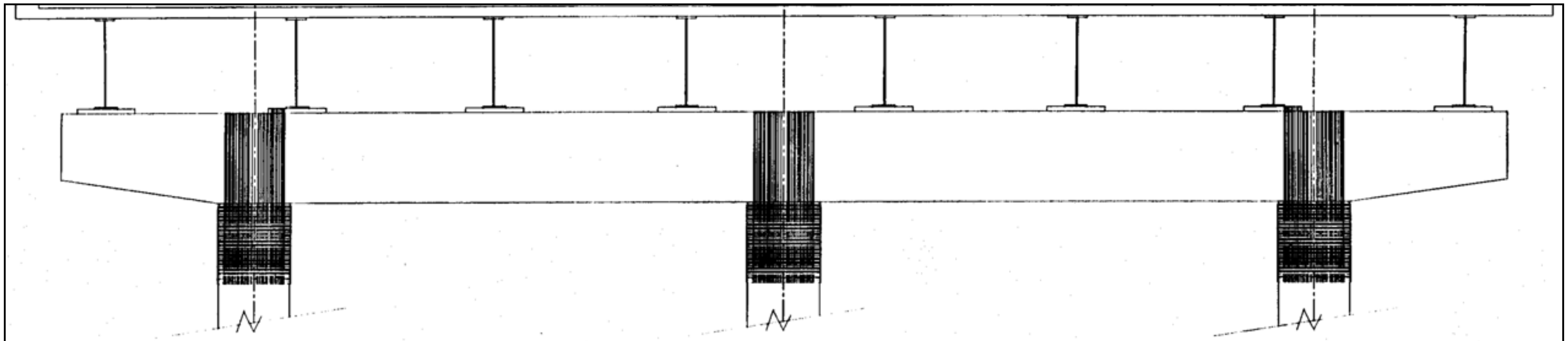
The goal of the seismic retrofit was to improve the displacement ductility of Bent #6 by a factor of two as compared to the as-is Bent #5. Structural analysis showed that the bent had deficiencies in the following areas: the confinement of the column lap splice region, the confinement of the plastic hinges, the column shear, the shear in the joint region, and the anchorage of the column longitudinal reinforcement into the cap beam. To address these issues, each element was analyzed and University of Utah researchers specified the structural retrofit using FRP composites.



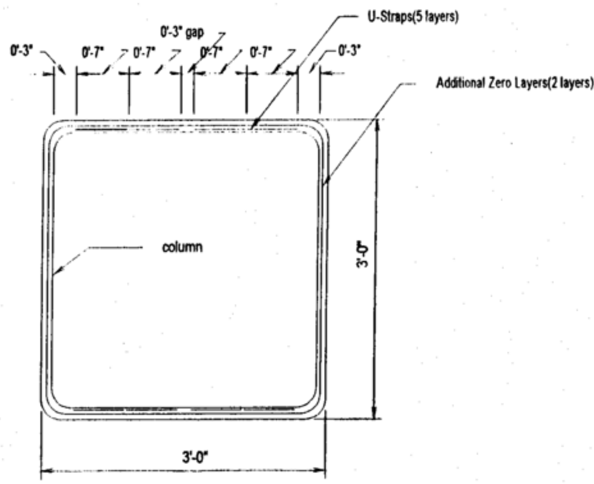


Retrofit Details

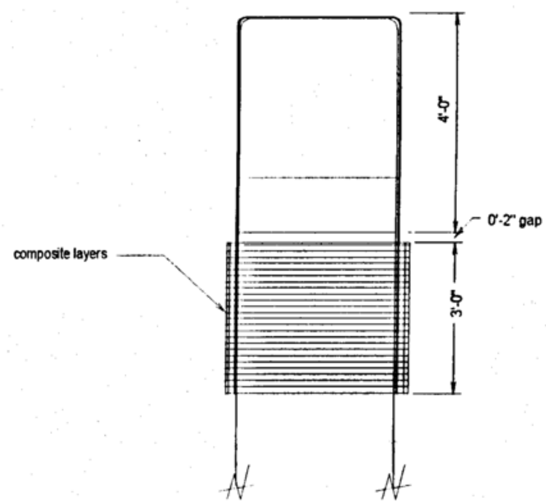
1) Layers = composite layers
2) All column composite layers are typical
3) Exterior composite joint layers are typical



U-Strap Top of Bent #6 Elevation



Plan View of Top of Column



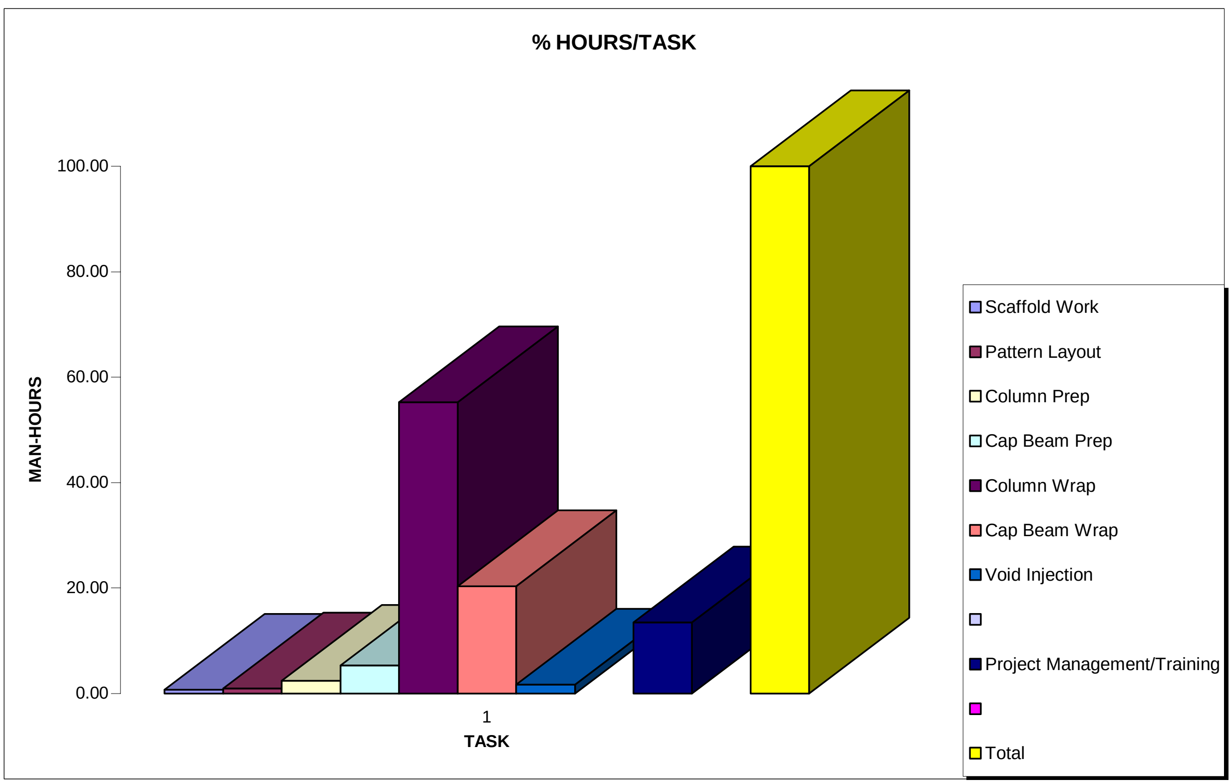
End View of Cap Beam

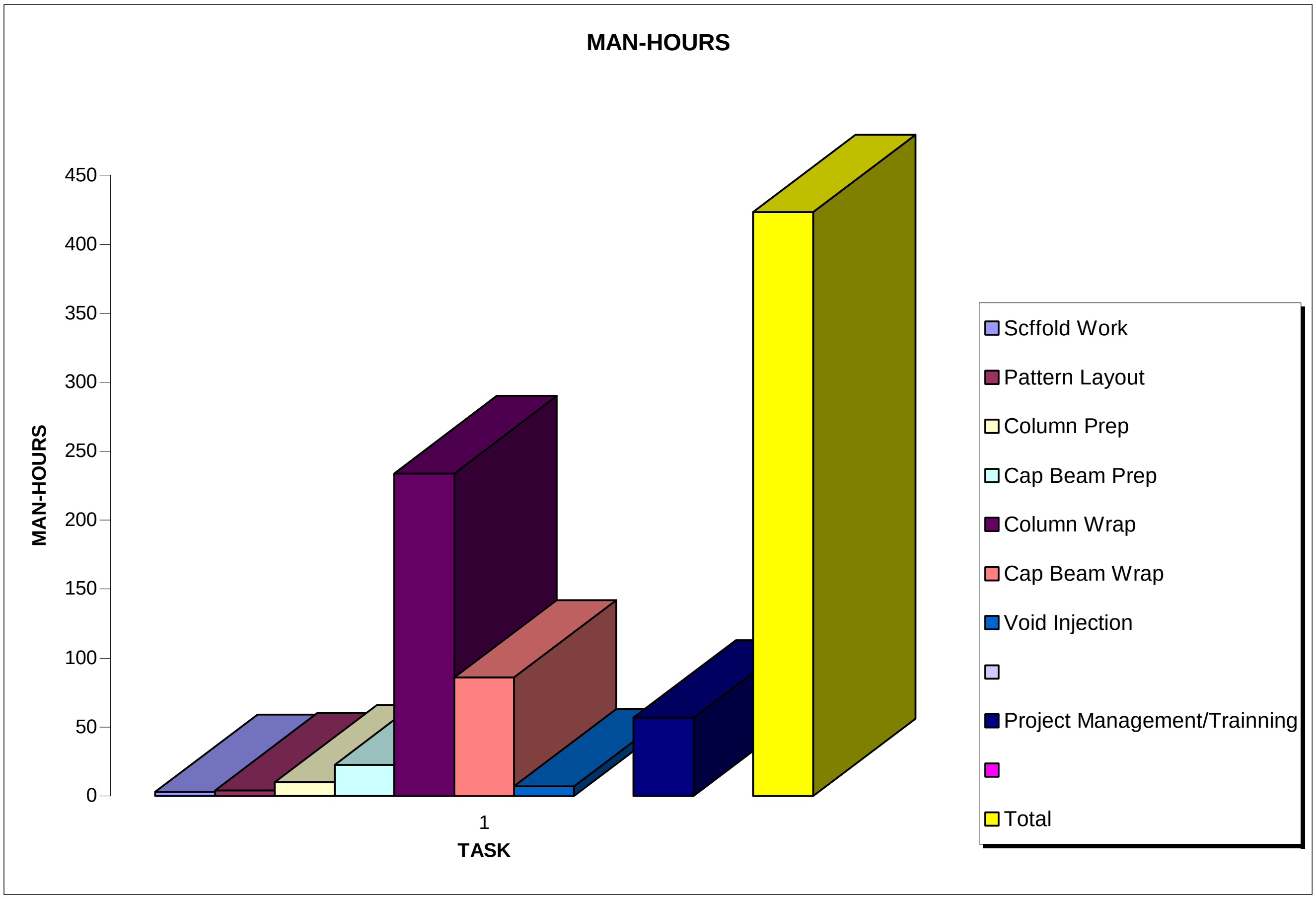
U-Strap Detail Sheet

Labor and Labor Analysis

The man-hours used, the labor analysis by hour and by a percentage of the total are given in the charts below.

Man-hour Breakdown							
Date/Task	**6/14/99**	**6/15/99**	**6/16/99**	**6/17/99**	**6/18/99**		**Total**
Scaffold Work	3						3
Pattern Layout	4						4
Column Prep	10						10
Cap Beam Prep	22.5						22.5
Column Wrap		106.5	91.5	36			234
Cap Beam Wrap				47	39		86
Void Injection			7				7





Material and Material Analysis
Sika Corporation supplied the materials for this project. The materials supplied for the project were:

Sikadur® 31 High-modulus, Structural Epoxy Adhesive
Sikadur® Hex 300 Impregnating Resin
Sikawarp® Hex 103C Carbon Fabric

In addition to the materials supplied by Sika Corporation, Hydrotech supplied the saturation machine and labor for the installation.

The following chart gives a breakdown of the materials and their usage.

	Lot Numbers	Column Total	Cap Beam Total	Sq. Ft. Column	Sq. Ft. Cap Beam
Resin System					
Sikadur® Hex 300 Impregnating Resin A	300A042299-1	17 Units	10 Units		
Sikadur® Hex 300 Hardener B	300B051999-2				
Density Sikadur® Hex 300 Impregnating Resin A	36.23 lb./ 2.78 gal/Unit	615.91	362.3		
Density Sikadur® Hex 300 Hardener B					
Total (Lbs.)		**615.91**	**362.3**		
Profiling Compound					
Sikadur® 31 Hi-Mod Gel A	A90002M	1	10		
Sikadur® 31 Hi-Mod Gel B	M80032M				
Density Sikadur® 31 Hi-Mod Gel A	29.5 lb./ 2 gal/Unit				
Density Sikadur® 31 Hi-Mod Gel B					
Pounds		29.5	295		
Fabric					
Sikawrap® Hex 103C					
	9C60116A (ft. x 18" Width)	2601	480	3901.5	720
	9C60116A (ft. x 7" Width)		1080		629.64
Total (Ft. Sq.)				**3901.5**	**1349.64**
Density Sikawrap® Hex 103C	18				

(Oz/sq. yd.)					
Weight of Sikawrap® Hex 103C Used (Lbs.)				487.69	168.71
Total (Lbs.)				**487.69**	**168.71**

Prior to the retrofit process, the University of Utah had the columns prepared for the wrap. This included hydro-blasting and rebuilding the surfaces of the cab beam with Shot Crete. Below are the costs associated with this phase of the project.

Operation	**Man hours**	**Cost**
Shot Crete	300	$10,000.00
Water Blast	67	$6,000.00
Scaffold		$2,000.00

Other miscellaneous charges incurred for the project are listed below.

Equipment	**Time**	**Cost**
Man Lift	One week	$850.00
Trash Removal	One week	$150.00
Toilet		$70.00

COLUMN PREPARATION

The first step for a successful retrofit job is to clean the concrete structure to ensure good adhesion between the adhesive and concrete. This was accomplished by using water jet blasting. Since the water jet was done one week before the columns were repaired, the surfaces had to be ground and brushed off to remove dust. The profiling material, Sikadur® 31 Hi-Mod, high strength structural adhesive, was used to patch large voids. All corners of the columns were rounded and smoothed using the grinders. After cleaning, patching and grinding, the columns were marked where the carbon fabric needed to be laid down. In the case of columns, the wrap started 2" from the top of the footing and 2" down from the column-cap joint.

Cleaning Column

Applying Sikadur® 31

Pattern Layout Prior to Repair

The final step in preparing the columns was to coat the area that would accept the carbon cloth with Sikadur® Hex 300 Impregnating Resin. This is the same resin that is used to impregnate the carbon cloth. The reason the resin is applied is to give the impregnated carbon cloth a better bonding surface. It should be noted that Sikadur® 31 structural adhesive was not used to facilitate bonding to the concrete substrate.

Base Resin Application

Once the base resin coat has been applied, the column is ready to accept the carbon composite material. The material selected by Sika was Sikawrap® Hex 103C carbon fabric in combination with Sikadur® Hex 300 Impregnating Resin.

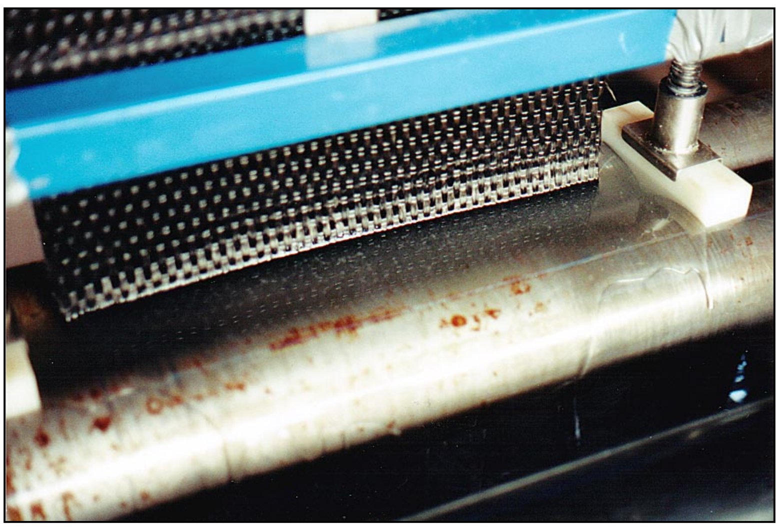
The carbon fabric for the columns was 18” wide and came in roll form. The resin came pre- weighed from the factory in two containers, one for part A and the other for part B. There is sufficient free volume in container A to accept all the contents of container B.

Mixing “A” & “B”

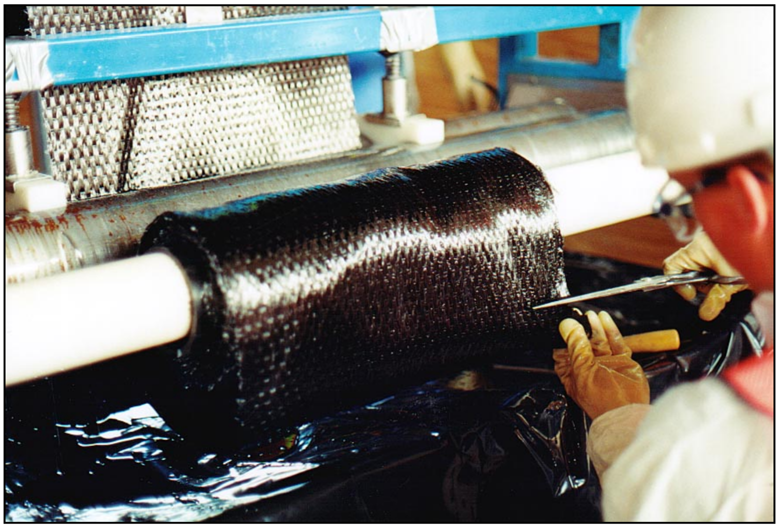
The contents of container “B” are poured into container “A” and the mixture is mixed for 3 to 4 minutes. This is the manufacture’s recommended procedure and corresponds to typical industry practices.

Once the resin has been mixed, the carbon fabric is impregnated. Sika used a saturation machine for this purpose. The saturator is a machine with two rollers that combines the resin and fabric. This method is very efficient. The saturator is calibrated for the resin viscosity, fabric weight, temperature and fabric width. The fabric is loaded on a let-off and passed through the two saturation rolls. This action forces the resin into the fabric and eliminates the possibility of having dry fibers appear in the finished composite jacket. The machine is manually operated and is suited for a construction environment. Once the desired length of fabric has been saturated, the material is cut off using scissors.

Saturation Unit



Impregnation Process



Cutting Material to Size

The impregnated carbon fabric is then taken to the column and applied. The application takes place on the column that has already had a layer of Sikadur® Hex 300 Impregnating Resin applied to the surface. It is important to note that Sika suggests that the layer of Sikadur® Hex 300 Impregnating Resin is still wet when applying the first layer.

The material was cut to a length sufficient to apply 5 layers in one continuous application step. This technique is typically referred to as a "jelly roll". The material was applied to one column at first that served as a training column then it was applied to the other two columns by two different teams.

The "jelly roll" method, although saving time, may have been partially responsible for some void problems that occurred in a later phase of this project.

Applying Material in "Jelly Roll" Fashion

This method of application was continued until the lower portions of the columns were completed. Once this had been accomplished, the upper sections of the three columns were completed according to the design.

Applying Carbon to Upper Portion of Column

The installation on the lower and upper portion of the columns was done in accordance with the manufacturers instructions. The first 5 layers were applied on one day and then allowed to cure. On the second day, the remaining layers were to be applied.

On the second day, the lower portions of the columns were inspected prior to the installation of the remaining layers of carbon. Upon inspection it was noticed that there were a number of voids found in the jackets particularly on the east and west columns that were exposed to direct sunlight during cure. These voids were of various sizes, from small (the size of a quarter) to very large (20 in^2 to 30 in^2). This defect had to be addressed prior to proceeding with the remaining wraps of carbon cloth.

Voids in Jacket

The void areas were repaired using the following procedure:

- Locate the void areas.
- Drill ~ ¼" diameter hole in the lower and upper portions of the void.
- Inject Sikadur® Hex 300 Impregnating Resin into the voids until it flows out the top hole.
- Plug the lower hole with Sikadur® 31.
- Allow the resin to cure.

Injecting Resin Into Void

Sikadur® 31 Patching Lower Hole

The void problem required 8 additional man-hours to repair. This must be avoided in future applications because of the cost that will be incurred by the contractor.

There are several possible reasons for the voids forming in the jacket. It should be noted however, that voids in a confinement application are not necessarily bad. Although this problem does not look good there should be enough strength in the repair area to insure a long enough development length to assure proper bonding between layers.

Possible reasons for void formation were:

- Jelly-roll installation process. This has been noted on other retrofit jobs.

- The resin, Sikadur® Hex 300, has a long gel and cure time. This could cause the fabric to “bag” under its own weight and cause layers to separate.
- The use of a structural adhesive as the first layer on the concrete was not done. Normally, flat surfaces, even on square columns, are treated with an adhesive prior to installation of the carbon material.
- Out-gassing from the concrete.
- Direct exposure to the sun during cure.
- Grinding was the surface preparation vs. sandblasting which would be preferable.
- Sikadur® Hex 306, which is a heavier, more thixotropic epoxy might have sealed the pores of the concrete better thus reducing out-gassing from the concrete.
- The use of a cementitious or epoxy leveling mortar on the columns before wrapping will reduce or eliminate concave or low spots and out-gassing of the concrete.

Things to do in future applications:

- Use of the fabric saturator is essential.
- Use of cementitious or epoxy leveling mortar on concrete before wrapping.
- Sandblast surfaces to receive composite wrap.
- Use of Sikadur® Hex 306 epoxy.
- Apply fewer wraps at a time.
- Shading composite from direct sunlight during cure.
- Covering composite with a cementitious or acrylic coating to protect it from ultraviolet light damage for long-term performance.

In order to assure the highest quality composite jacket installation it is important for the contractor and the material supplier to adjust their installation procedure to avoid the formation of voids in the future.

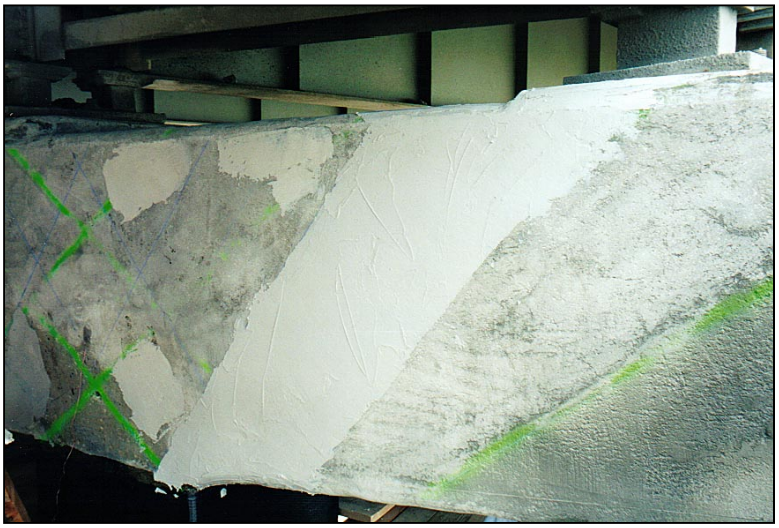
CAP BEAM PREPARATION

The first step for a successful cap beam retrofit job is to clean the concrete structure to ensure good adhesion between the adhesive and concrete. This was accomplished by grinding the entire surface of the Shot Crete that would receive the composite material. The profiling material, Sikadur® 31 Hi-Mod, high strength, structural adhesive was used to patch large voids. Sikadur® 31 Hi-Mod, high strength, structural adhesive was also used as the prime (coat between the concrete and the first layer of carbon material) layer on the cap beam. All corners on the cap beam were rounded and smoothed using the grinders. After cleaning, patching and grinding the cap beam was marked to indicate the placement of the carbon fabric.

Procedure for the cap beam differs from that of the column wrap. All steps including marking the columns, applying the adhesive, carbon and resin are the same. The only difference is in wrapping the 52 degree and 90 degree pieces around the cap. The +52 degree pieces are laid down first. The fabric is started from the mid-face of cap at top to mid-face of cap at bottom. The –52 degree fabric is then laid down overlapping the previous +52 degree piece. Once the 52 degree pieces are in place, the 90-degree hoops are wrapped as close to the joint as possible. These start and end at the mid-face of the bottom of cap. The thin 6" straps are then placed as shown in the design. These begin on the column, wrap all the way across the cap and then end on the opposite face of the column. Final two layers of carbon are placed around the straps on the column surfaces to secure the straps in place.

Sikadur® 31 Applied to Fill Uneven Spots

Sikadur® 31 Applied as Prime Layer to Cap Beam



Sikadur® 31 Applied to Bottom of Cap Beam

This sequence of pictures is very important. The adhesion between the concrete substrate and the composite material is vital to the overall strength of the system. The loads must be between the concrete and the composite through this layer. If there are any voids on the flat surfaces the loads will not be transferred properly, thus reducing the overall strength of the system.

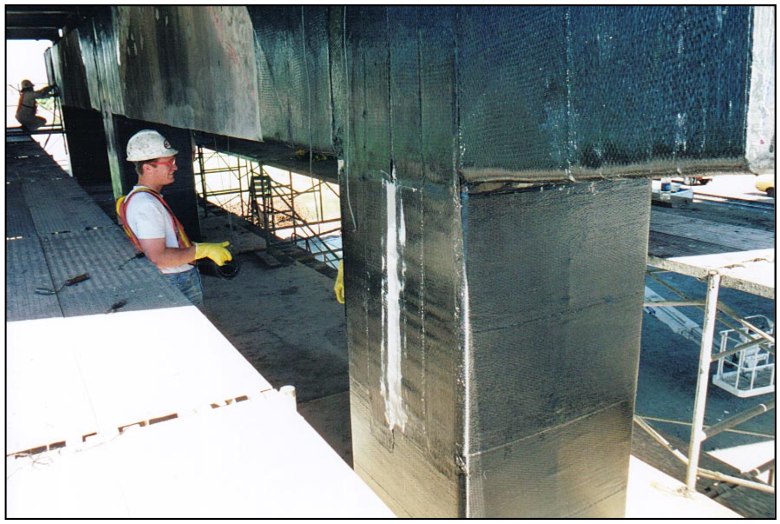
52 Degree Layer Being Applied

52 Degree Layer In Place

0 Degree Layer Being Applied

0 Degree Layer In Place

Application of Six Inch Strips



Six-Inch Strips In Place

Final Layer Applied to Column to Tie 6 in. Strips Down

Avoid starting from the corners. While wrapping the first layer of carbon, one person lays the fabric down and a second person consolidates using a metal roller. This ensures uniform bond to adhesive and minimum air pockets. Make sure there is a 6” overlap at the end.

SAFETY

All people working on the project were schooled on the safety measures expected on the project.

Railroad safety training was mandatory for everyone on the project. Use of fall protection equipment was also emphasized. Most important was the training on the use of epoxy materials and the protective gear needed to work on the project.

Safety procedures were reiterated on a daily basis. The UDOT safety officer audited the site several times during the duration of the project.

Assign individuals for each column; mixer, resin applicator, adhesive applicator and impregnator/consolidator, safety officer/supervisor/quality assurance was successful and each group knew and understood safety related to each job function.

Safety meetings were held at the beginning of each shift. Each project leader was responsible for the safety of their crews.

4.CONDITION OF THE I-80 STATE STREET STRUCTURE

State Street Bridge at I-80 in Salt Lake City was designed in 1965 according to the State of Utah Standard Specifications for Road and Bridge Construction, 1960 Edition and Supplements, and the AASHO Specifications of 1961 and Interim Specifications. As such, the bridge was not designed to resist earthquake-induced forces or displacements; only wind loads were considered. The 55.34m-long bridge consists of a 10.82m end span, a 10.69m end span, and a 33.83m middle span, as shown in Fig. 4.1. The steel composite welded girders are simply supported at the abutments, which are of the seat type, and at the bents. The substructure consists of four cast-in-place bents, two on the east and two on the west side as shown in Fig. 4.2; each of the bents consists of four circular columns and a bent cap. Each column is supported by a concrete pile cap with four piles as shown in Fig. 4.3. The dimensions and reinforcement of one of the bents is shown in Fig. 4.3; the three middle bottom bars (36mm) in section A-A are bent up near the exterior column and shown on top in section C-C.

Design Spectra for three earthquakes were determined as follows: (1) 0.2 g earthquake, or a 10% probability of exceedance in 15 years, which represented the design life of the retrofit because the east I-80 corridor in Salt Lake City was scheduled for replacement by 2015; (2) 10% probability of exceedance in 50 years; (3) 10% probability of exceedance in 250 years. The design spectra shown in Fig. 4.4 were used for retrofit analysis and design; they were obtained from the I-15 design spectra and were adjusted for local soil conditions (Steven Bartlett, personal communication, February 1999).

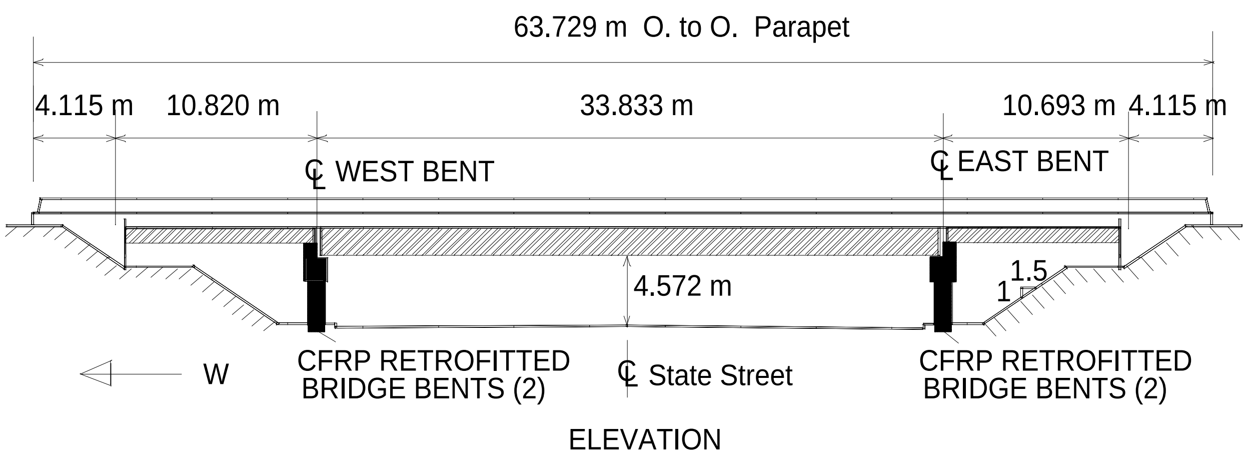


FIGURE 4.1. STATE STREET BRIDGE ELEVATION SHOWING THE FOUR BENTS RETROFITTED WITH CFRP

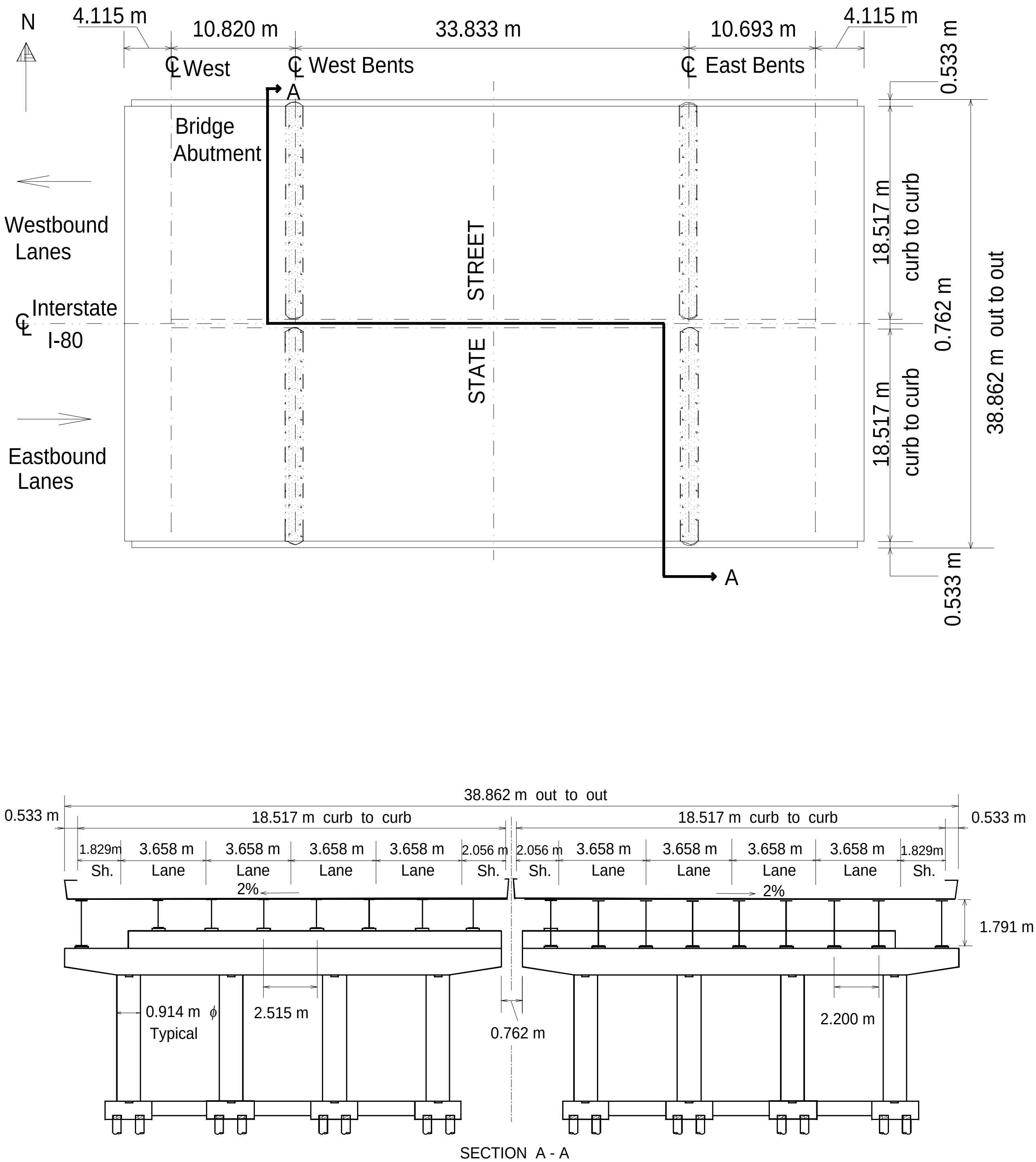


Figure 4.2. Plan and sectional view of State Street Bridge

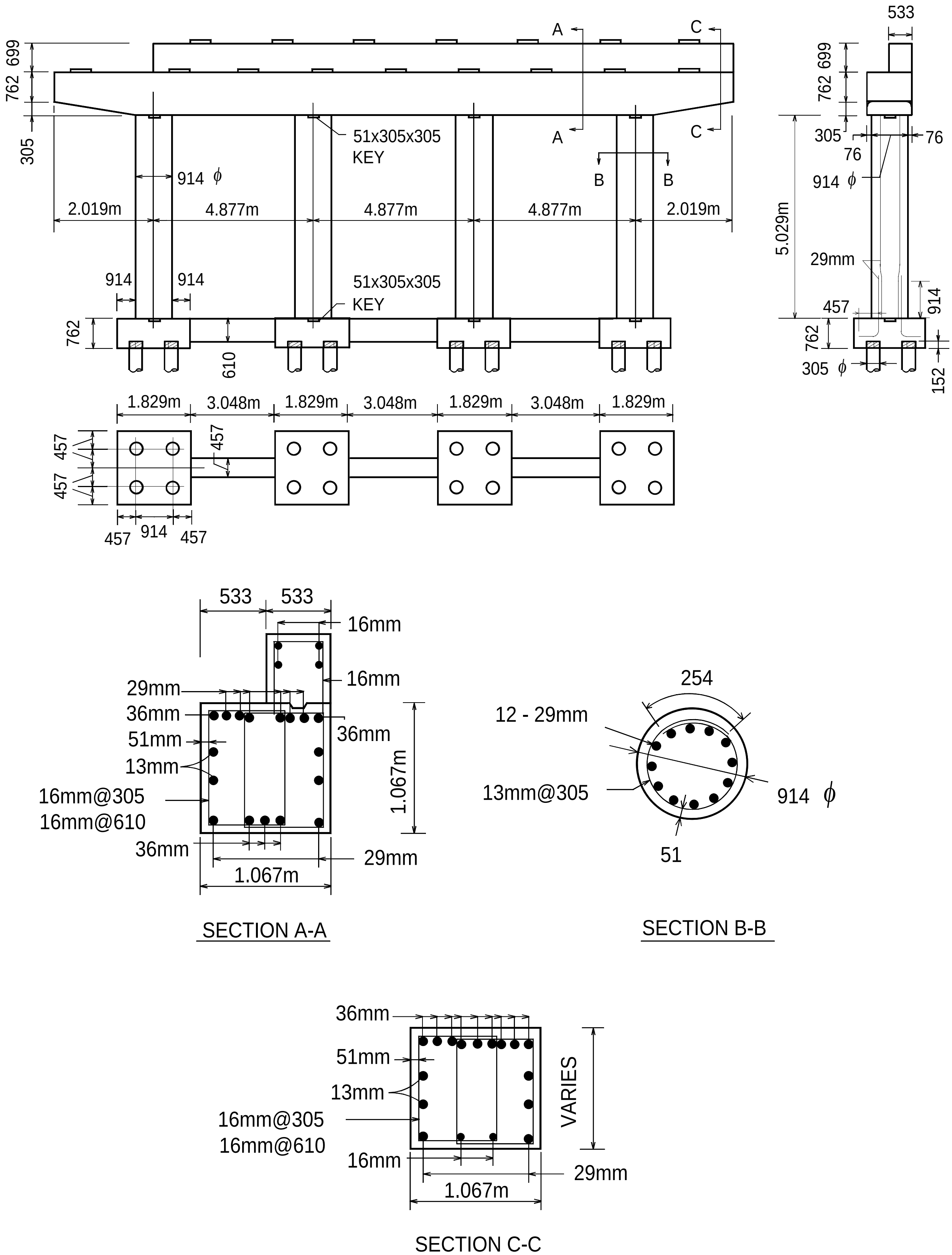


Figure 4.3. Dimensions and Reinforcement of Typical Bent of State Street

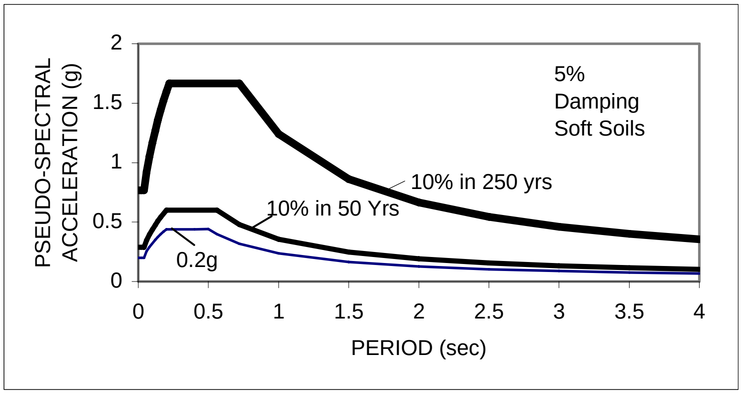


Figure 4.4. Design spectra for seismic retrofit of State Street Bridge

BRIDGE IN THE AS-BUILT CONDITION

A capacity evaluation was performed with these criteria: (a) buckling of longitudinal column reinforcement, (b) crushing of concrete core at plastic hinge regions and maximum curvature capacity, (c) principal stresses at bent cap to column joints, (d) anchorage of longitudinal column bars in the bent cap, (e) shear, axial, and flexural capacity of columns and bent cap, (f) lap splice failure at column to foundation connection, and (g) principal stresses at column to pile cap connections. Static pushover nonlinear analyses of the bridge bent were performed in the longitudinal and transverse direction; the two-dimensional model used in the transverse direction is shown in Fig. 4.5. The structure was modeled using DRAIN-2DX (Prakash et al. 1992), for the as-built and retrofitted bents (Pantelides et al. 1999b). Gravity loads were applied to the bent from the superstructure, and a moment curvature analysis of the columns with the axial load applied was carried out. The pushover analyses identified the capacities; based upon the structural response to the earthquake, the demands were determined and compared. The capacities were based upon the limiting strains and stresses for reinforcing steel and concrete (California Department of Transportation 1998).

Design parameters for analysis required in the seismic retrofit were established as follows: (1) unconfined concrete compressive strength = 29 MPa, (2) maximum unconfined concrete strain = 0.004, (3) steel yield stress: 300 MPa, and (4) foundation lateral stiffness in horizontal direction = 28.3 kN/mm, axial stiffness in vertical direction = 168.1 kN/mm, and rotational stiffness = 45,685 kN-m/rad.

Cap Beam and Columns

The pushover curve of the as-built bent in the transverse direction is shown in Fig. 4.6 as a solid line; the global displacement ductility is $\mu = 2.9$. Table 4.1 shows a summary of displacement demands, and column flexure and shear demand/capacity ratios, obtained using the X-Section and W-frame programs developed by Caltrans (California

Department of Transportation 1998). These results revealed column flexural and shear force deficiencies for the 10% in 250 years hazard level.

The pushover curve of the as-built bent for the longitudinal direction in Fig. 4.7 shows the effect of a very flexible footing associated with a weak rotational spring; this allows the columns to ride the earthquake with very little plastic action, thus engaging the abutment lateral stiffness. This behavior required addition of a bumper bracket at the bottom of the steel girders to ensure positive engagement of the abutment stiffness in the longitudinal direction. Bearings and end diaphragms were checked and found to be adequate for the load path examined. The largest slenderness factor kl/r was found to be equal to 58; this was adequate for seismic action and is considered relatively strong for same vintage structures where the kl/r factor is typically substantially larger.

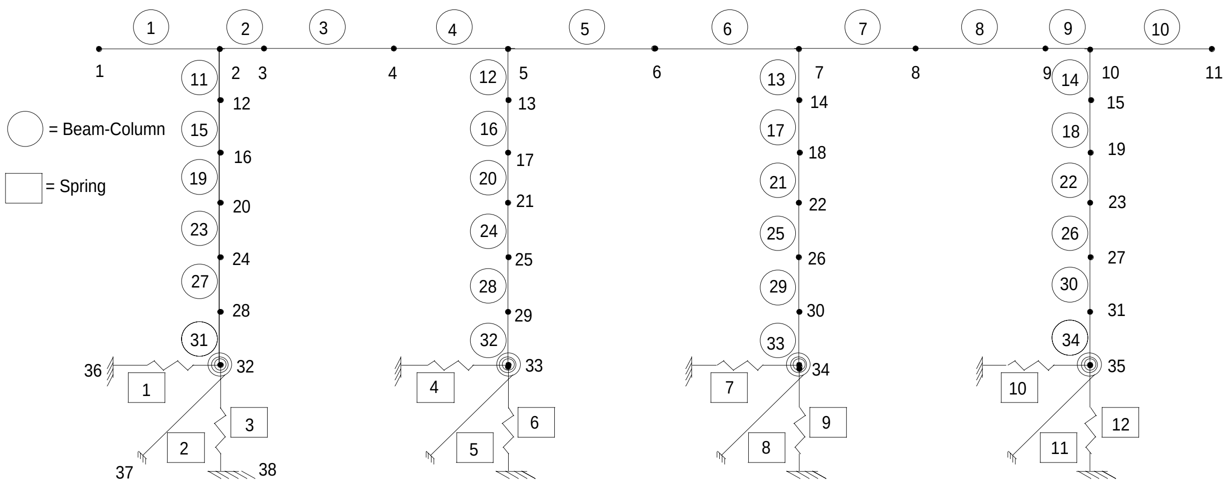


Figure 4.5. Structural model of State Street Bridge bent for analysis in the transverse direction

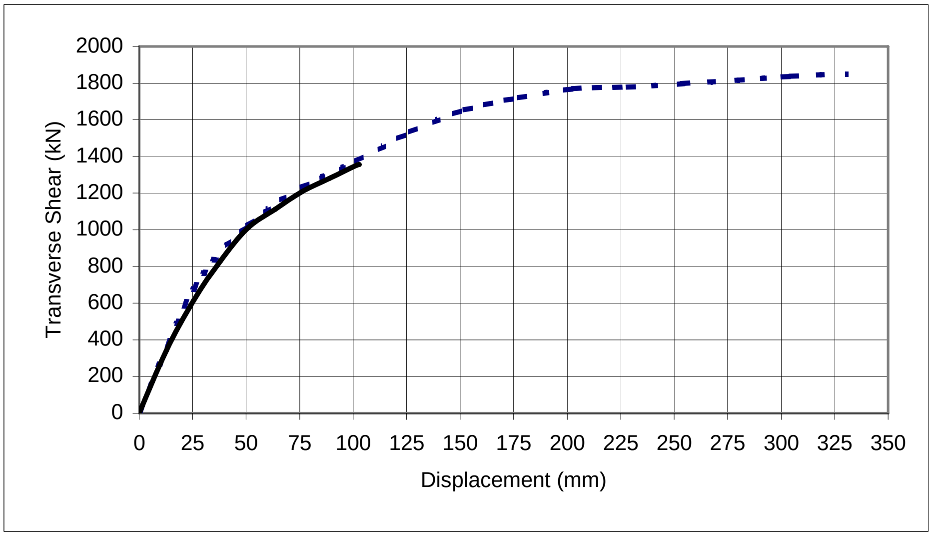


Figure 4.6. Pushover curve for the bent in the as-built and retrofitted condition for the transverse direction (solid = as-built, dotted = retrofitted)

Table 4.1. Displacement Demands and Demand/Capacity Ratios

Design EQ	0.2g (Service Level)	10% in 50 yrs (No Collapse)	10% in 250 yrs (No Collapse)
PSA	0.33 g	0.79 g	1.667 g
Δ_{demand}	39 mm (1.54")	58 mm (2.30")	199 mm (7.83")
Δy	37 mm (1.47")	37 mm (1.47")	37 mm (1.47")
Δultimate	60 mm (2.36")	60 mm (2.36")	60 mm (2.36")
μ_{demand}	1.04	1.56	5.33
$\mu_{capacity}$	1.61	1.61	1.61
Flexure d/c Ratio	0.65 Essentially Elastic	0.97 Marginal	3.44 N.G. for Flexure
Shear d/c Ratio	<1.0 elastic	0.57 Acceptable	5.07 N.G. for Shear

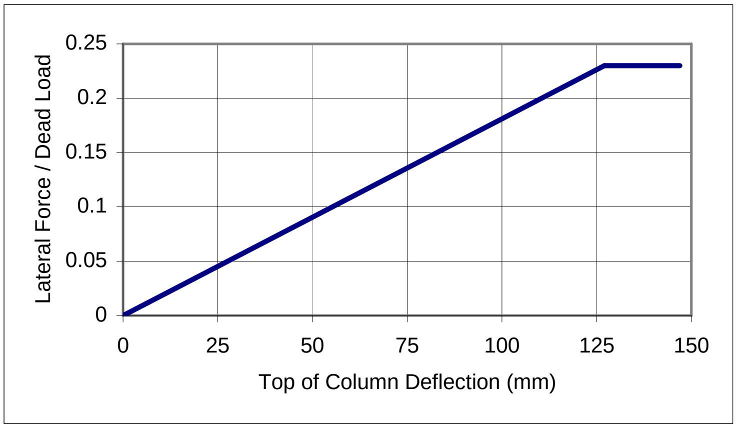


Figure 4.7. Pushover curve in longitudinal direction for bent in the as-built condition

Footings

The footing details were inadequate for the desired seismic performance, and anchorage of longitudinal reinforcement in the pile cap was insufficient. In addition, shear reinforcement at the top of the pile cap, embedment of reinforcement from the piles to the pile cap, size and reinforcement of the grade beam connecting the pile caps, and reinforcement in the pile cap joints were inadequate. However, these inadequacies did not increase the vulnerability because of the seismic retrofit approach that was adopted. Recent analytical work and experience from previous earthquakes are permitting a new approach for treating these foundation deficiencies. The seismic retrofit approach adopted is that the bottom of the columns will allow rotation or uplift for a fixed connection. A stable rocking behavior of the bent is used rather than tie-downs or additional piles to accommodate earthquake lateral demands (Alameddine and Imbsen 2002).

5. Training

University of Utah instruction to UDOT

The opportunity to apply carbon FRP composites to existing bridges in Salt Lake City for strengthening as well as seismic retrofit, created the need for training of structural engineers at UDOT in these matters. Professor Chris Pantelides from the Civil & Environmental Engineering Department of the University of Utah provided several lectures regarding the following topics to a group of UDOT Engineers from the Structures Division:

1. Pushover nonlinear analyses of reinforced concrete bridge columns. These included columns in the as-built condition and columns retrofitted with carbon FRP composites. The nonlinear analysis program DRAIN-2DX developed at the University of California, Berkeley was used.

2. Pushover nonlinear analyses of reinforced concrete bridge bents. These included multicolumn and single column bents in the as-built condition and bents retrofitted with carbon FRP composites. The nonlinear analysis program DRAIN-2DX developed at the University of California, Berkeley was used.

3. Design of carbon FRP composite retrofit of columns for strength. These columns were outdated based on their progressive corrosion problems and the new live loads on today's interstate highways that are larger than the design loads they were designed for almost 40 years ago.

4. Seismic retrofit design of reinforced concrete bets using FRP composites. These bridges were originally designed for smaller gravity loads than what exists in today's interstate highways; in addition they were designed only for wind loads but not for earthquake loads. With today's awareness of the potential for a large earthquake from the Wasatch fault it was decided to perform a seismic retrofit design of one bridge, the State Street Bridge on Interstate 80 in Salt Lake City.

5. Supervision and checking of retrofit designs for strengthening of columns with carbon FRP composite for the Foothill Drive Bridges, by UDOT Engineers from the Structural Division.

6. DESIGN

Several strategies exist for seismic retrofit of older RC bridges, such as State Street Bridge. The first strategy would be to use steel jackets for shear and flexure in the columns, and a bent cap retrofit using concrete jacketing with through bolts to increase confinement. This strategy was not selected because of concern for the time required for construction of the bent cap concrete jacket. A second strategy would be infill walls between columns; this strategy was rejected, since it does not take care of column inadequacies in the longitudinal direction. As an alternative to steel column casings the use of CFRP jackets was considered. Currently, Caltrans standard practice does not include the use of FRP wraps to seismically retrofit bent caps or bent cap-column joints (California Department of Transportation 1998).

Based on in-situ tests (Pantelides et al. 1999a, 2001, 2002a, 2002b), a CFRP composite retrofit, including the bent cap and bent cap-column joints was adopted since it caused minimum traffic disruption and could be completed in the shortest time. In addition, CFRP composites are lightweight, resist electrochemical corrosion, and can assume practically any geometrical shape. The choice of an FRP composite seismic retrofit was encouraged by the Federal Highway Administration as an application of innovative technology. Other retrofit measures were also implemented; to reduce future maintenance requirements, the deck was made continuous over the expansion joints. The bridge abutments were of the seat type; bumper brackets at the bottom of every steel girder were used to ensure positive engagement of abutment stiffness in the longitudinal direction, as shown in Fig. 6.1.

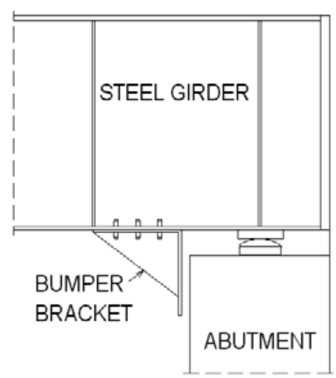


Figure 6.1. Bumper bracket details

RETROFIT OF COLUMNS

The seismic retrofit goal was to improve the bridge's displacement ductility and seismic load resistance. From a vulnerability analysis, the bents were found deficient in: (a) confinement of column lap splices, (b) confinement of column plastic hinges, and (c) shear in the columns, bent cap, and bent cap-column joints. To address these issues, each element was analyzed and structural retrofit using CFRP composites was specified. The structural analysis variables used in the CFRP design were obtained from the DRAIN-2DX model (Pantelides et al. 1999b).

Flexural Plastic Hinge Confinement of Columns

To confine the plastic hinge region, the CFRP composite was designed as a circular jacket. The thickness of the CFRP layers was (Seible et al. 1997):

$$t_j = \left[\frac{0.09D\ (\varepsilon_{cu} - 0.004) f'_{cc}}{\phi_f f_{ju} \varepsilon_{ju}} \right] \tag{6.1}$$

where the column diameter D = 0.914m; ε_{cu} = required ultimate concrete strain, taken as 9.6 mm/m based on the required ductility increase; this was targeted to a retrofitted bent displacement ductility of μ = 5; f'_{cc} = compressive strength of confined concrete taken as 1.5 times the compressive strength of unconfined concrete; f_{ju} = ultimate CFRP composite strength evaluated according to ASTM D-3039 specifications as 630 MPa; ε_{ju} = ultimate composite strain evaluated as 10mm/m; and ϕ_f = flexural capacity reduction factor taken as 0.9. The material used in this application was 12,000 individual unidirectional carbon fibers per fiber bundle (tow), approximately 5 to 7 microns in diameter. There were 19 tows per 25mm width of fabric yielding a total of 228,000 fibers. The width of the fabric sheet used was 635mm. The required CFRP composite thickness from Eq. (6.1) was 3.6mm.

Column Lap Splice Clamping

The CFRP composite thickness required for clamping the lap splice region was determined from the difference between the lateral clamping pressure required to maintain the bond, and the contribution of steel hoops. The lateral clamping pressure is (Priestley et al. 1996):

$$f_l = \frac{A_s f_{sy}}{\left[\frac{p}{2n} + 2(d_b + cc) \right] L_s} \tag{6.2}$$

where A_s = area of one longitudinal column reinforcing bar (645mm^2); f_{sy} = yield strength of column bars; p = inside crack perimeter along the lap-spliced bars (2.294m); n = number of column bars (i.e. twelve); d_b = diameter of bars (29mm); cc = concrete cover to longitudinal bars (63.5mm); and L_s = lap splice length (914mm); Eq. (6.2) gives f_l = 765 kPa. The contribution of steel ties to the clamping force is:

$$f_h = \frac{0.002 A_h E_h}{Ds} \tag{6.3}$$

where A_h = area of transverse ties (129mm^2); E_h = elastic modulus of ties (200 GPa); D = column diameter (914mm); and s = spacing of ties (305mm); the resulting stress was found as f_h = 186 kPa. Based on the values from Eqs. (6.2) and (6.3), the CFRP composite thickness to clamp the lap splice region was (Seible et al. 1997):

$$t_j = \left[\frac{500D\ (f_l - f_h)}{E_j} \right] \tag{6.4}$$

where E_j, the CFRP composite elastic modulus was determined experimentally as 65 GPa; the required CFRP thickness was 4.1mm. Outside the plastic hinge region, assuming a minimum confinement pressure of 1,034 kPa, yielded a CFRP composite thickness of 1.8mm.

Shear Strengthening of Columns

Each of the shear resisting components was subtracted from the design shear, which was taken as 1.5 times the column shear at yield. No shear strengthening was necessary outside the plastic hinge region. The CFRP jacket thickness inside the plastic hinge region was (Seible et al. 1997):

$$t_j = \frac{159\left[\dfrac{V_0}{\phi_v} - (V_c + V_s + V_p) \right]}{E_j D} \tag{6.5}$$

where V_0 = design shear estimated at 472 kN; V_c = shear contribution of concrete (116 kN); V_s = shear contribution of ties (138 kN); V_p = effect of axial load (18 kN); D = column width (914mm); and ϕ_v = shear strength reduction factor (0.85). The required thickness was 1.0mm inside the plastic hinge region.

The pushover analysis for the retrofitted bent with CFRP composites is shown in Fig. 4.6 as a dotted line. The displacement ductility is μ = 5 corresponding to a displacement of 325mm. The column CFRP jacket design was obtained as shown in Fig. 6.2, where n = number of layers; each layer had a thickness of 1.32mm with the assumed CFRP composite properties. The required number of layers was combined for the effects of flexural confinement, lap splice clamping and shear strengthening. A 51mm gap between the column and pile cap and column and bent cap were left to avoid any strength and stiffness increase from the retrofit.

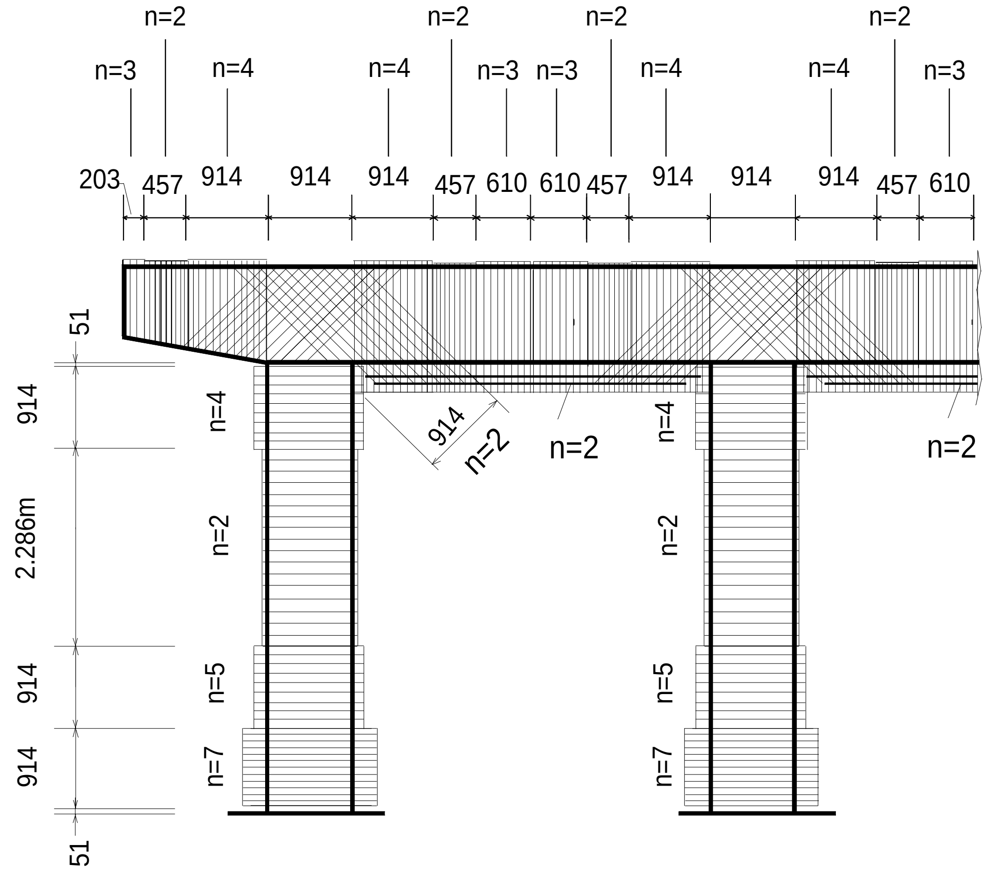


Figure 6.2. CFRP composite design for columns, bent cap, and joint "ankle wrap"

RETROFIT OF BENT CAP

Visual inspection of the bridge revealed delamination of the concrete cover at the bent cap. Delaminated concrete must be removed and replaced by shotcrete to achieve a good bond between CFRP composite and concrete (Pantelides et al. 2001). The CFRP composite design for the bent cap was based on analysis of the bent with CFRP applied on the columns, as calculated previously.

Flexural Strengthening of Bent Cap

At a lateral displacement of 325mm the bent cap steel yields in the positive moment region, in elements 3 and 8 (see Fig. 4.5). The positive moment capacity of the RC bent cap was less than the demand by 418 kN-m, at a lateral displacement of 325mm.

Therefore, flexural strengthening of the bent cap using CFRP sheets was required, applied at the bottom of the bent cap, with the carbon fibers placed parallel to the bent cap axis. The CFRP composite thickness required was (Pantelides and Gergely 2002):

$$t_j = \frac{T}{\varepsilon_{jf} \mathrm{E}_j w_j} \tag{6.6}$$

where T = required tension force in the CFRP composite (783 kN), E_j = 65 GPa, w_j = bent cap width (914mm) and ε_{jf} = tensile strain that can be developed in the CFRP composite, which was assumed as $0.8\varepsilon_{ju}$ or 8mm/m; the required CFRP thickness was 1.8mm. Two 914mm-wide layers were used as detailed in Fig. 6.2.

Shear Strengthening of Bent Cap

The bent cap concrete shear capacity was V_c = 300 kN. The contribution of stirrups from the two interior legs and one half of the exterior leg on each side, for a total of three legs was considered. The reason for not including the total area of exterior stirrups is electrochemical corrosion, which was evident from bridge inspection. The shear capacity due to stirrups was V_s=560 kN, for a total shear capacity of 860 kN. The demand was taken as 1.5 the shear at yield or the shear at the ultimate displacement of 325mm; the latter was 1,125 kN at node 10 of element number 9 (see Fig. 4.5). The CFRP composite thickness was found from Eq. (6.5) ignoring the axial component, for both the haunches and the region within the columns as 0.8mm.

Flexural Plastic Hinge Confinement of the Bent Cap

Since yielding occurred in the bent cap, plastic hinge confinement was considered. Equation (6.1) was applied, with an ultimate concrete strain of 7.8mm/m at a lateral displacement of 325mm. Since the bent cap had a rectangular cross-section, an equivalent circular diameter of 1.509m was used from the average of the oval jacket principal radii (Seible et al. 1995). The resulting CFRP jacket thickness in the bent cap hoop direction was 4.1mm, which was provided in the positive and negative moment zones for a width of 914mm. The design shown in Fig. 6.2 was obtained for the CFRP jacket in the hoop direction of the bent cap, where n = number of CFRP layers; the jackets were bonded to all four faces of the bent cap.

The bent cap CFRP composite design in Fig. 6.2 has a rounded-off thickness with a thickness layer of 1.32mm. In wrapping the bent cap, where conflicts with existing shoes were found, CFRP splices were used; fitting requirements were detailed on construction drawings. Surface preparation is important for the bent cap because the bond between the CFRP composite and concrete is critical. Issues that must be considered prior to application of the CFRP composite include: substrate repair including corroded steel, profiling of concrete surface using shotcrete, crack injection, chamfering of corners, concrete surface cleaning, and application of bonding agent. Detailed construction requirements are described by Pantelides et al. (2003).

RETROFIT OF BENT CAP-COLUMN JOINTS

Bent cap-column joints with typical pre-1960 design details do not exhibit satisfactory performance (Sritharan et al. 1999, Pantelides et al. 1999a). The CFRP composite design for the bent cap-column joints was based on analysis of the bent with CFRP applied on the columns and bent cap as calculated previously.

Shear Strengthening of the Bent Cap-Column Joints

The joint shear forces were evaluated to design the CFRP composite thickness in the joint region. This was achieved by modeling the retrofitted bent with CFRP composites, using DRAIN-2DX (Pantelides et al. 1999b). The confined concrete properties were determined using established procedures (Gergely et al. 1998; Moran and Pantelides 2002a, 2002b). From the pushover curve of the retrofitted bent, shown as a dotted line in Fig. 4.6, it can be seen that the peak lateral load was increased compared to the as-built bent, which resulted in higher joint shear forces. The calculated stresses in the joint at the ultimate displacement were: (a) joint shear = 1,805 kPa; (b) axial stress in vertical direction = 2,365 kPa; and (c) horizontal axial stress in bent cap = 415 kPa. The resulting principal stresses were: (a) tension σ_2 = 3,440 kPa, and (b) compression σ_1 = 660 kPa. The principal angle was calculated as 31^0 from the bent cap axis.

The joint forces, stresses, and principal stresses were found using established procedures (Pantelides and Gergely 2002). The fiber orientation was selected as $\pm 45^0$ from the bent cap axis for ease of construction. The joint principal tensile stress by carrying out the CFRP retrofit was increased by $\Delta\sigma$ = 525 kPa, from 2,915 kPa for the as-built bent to 3,440 kPa for the retrofitted bent.

To find the number of composite layers required to provide a higher shear capacity, a diagonal tension crack in the joint was analyzed as shown in Fig. 6.3. The crack direction was assumed perpendicular to the angle at which the CFRP composite unidirectional fibers were applied, in this case at 45^0. The force F_2, acting normal to the crack, resisted by one composite layer stressed in tension is (Pantelides and Gergely 2002):

$$F_2 = t\varepsilon_f E_j \frac{d_e}{\cos\theta_p} \tag{6.7}$$

where θ_p = angle between the member axis and fiber direction (45° in Fig. 6.3); t = thickness of FRP sheets (1.32mm); ε_f = average axial strain in the fiber direction at peak horizontal load (assumed as 2mm/m), which is a lower bound of strain gage measurements on CFRP composites in in-situ tests of similarly retrofitted bridge bents (Pantelides et al. 1999a, 2001, 2002b); E_j = elastic modulus of CFRP composite; and d_e = effective joint depth, which is the bent cap height minus twice the CFRP effective bond length to concrete (Fig. 6.3); from previous studies the effective bond length was approximated as 76mm. The bent cap dimensions and inclination of CFRP composite unidirectional fibers control the joint effective depth d_e, which was 914mm, as shown in Fig. 6.3; the resulting force F_2 was 222 kN.

To find the tensile stress in one composite layer, F_2 is divided by the joint width (b = 1,067mm) and by the inclined length (along the crack) as:

$$\sigma_f = \frac{F_2 \cos\theta_p}{bd_e} \tag{6.8}$$

This calculation yields σ_f = 158 kPa; enough layers, each of a capacity σ_f, were used to resist the stress increase beyond the cracking stress ($\Delta\sigma$ = 525 kPa) from the as-built to the retrofitted bent. The total number of layers required was:

$$n = \frac{\Delta\sigma}{\sigma_f} \tag{6.9}$$

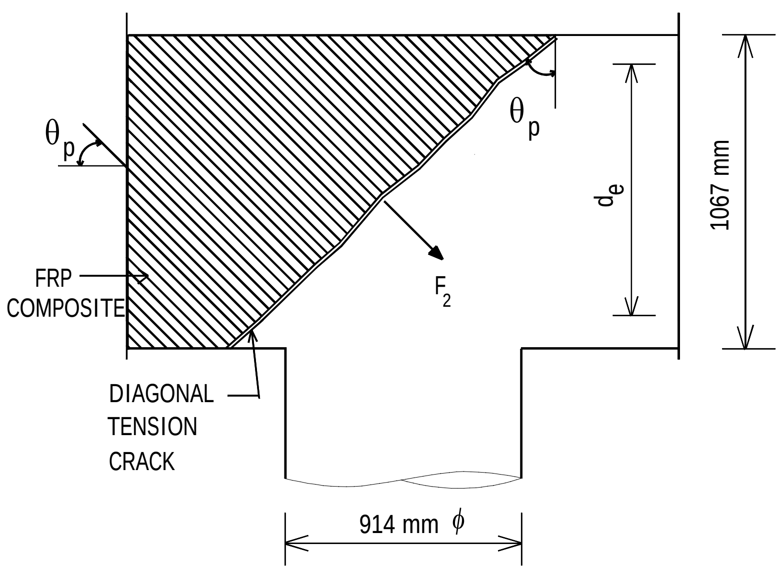


Figure 6.3. CFRP composite “ankle wrap” design parameters

for a thickness of 3.3 CFRP composite layers. Two 914mm-wide CFRP composite layers were provided on both sides for a total of 4 layers; the CFRP composite layers were applied in both directions (+45° and -45°) to account for cyclic earthquake loads; this forms the "ankle-wrap" scheme shown in Fig. 6.2. The bent cap corners were rounded to 51mm to provide better anchorage and reduce stress concentrations.

U-strap

To improve anchorage of longitudinal column bars ending in the bent cap, a "U-strap" CFRP composite scheme was implemented. The CFRP composite thickness was determined using Eq. (6.6). The tensile force in the columns of the retrofitted bent was determined using the analytical model as T = 1,230 kN. The CFRP strap effective width on each side of the U-strap was estimated as 710mm. The effective strain was assumed as 50% of the CFRP ultimate strain, or 5mm/m, to avoid premature tensile failure; this required a thickness of two layers as shown in Fig. 6.4. The U-straps were brought down 305mm from the bent cap bottom and extended 635mm on the column, to avoid stress concentrations; an additional CFRP layer was applied around the column to clamp the strap to the column. Recent in-situ tests of RC bridge bents using a similar CFRP composite seismic retrofit showed that the flexural overstrength provided by the CFRP U-straps was 18% of the lateral capacity of the as-built bent (Pantelides et al. 2002a); the design details of the present retrofit are adequate to handle overstrengths of this magnitude. The gap left between the strap, bent cap and column was filled with structural foam as shown in Fig. 6.4. The final CFRP composite retrofit for one of the bents is shown in Fig. 6.5. The U-strap details before and after a protective paint cover was applied are shown in Fig. 6.6.

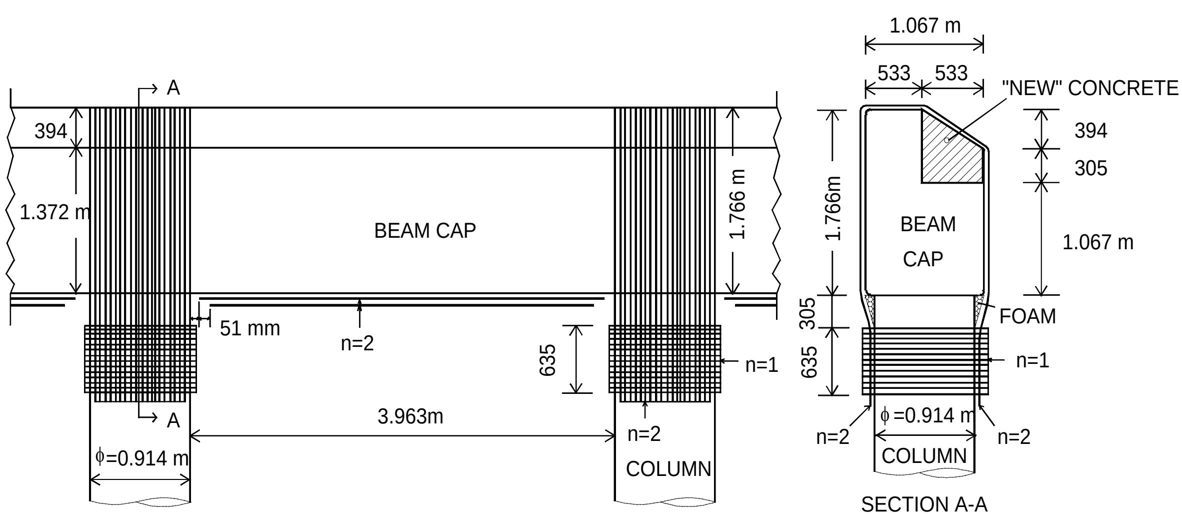


Figure 6.4. CFRP composite design for column to bent cap "U-strap"

Figure 6.5. CFRP composite application on State Street Bridge bent

(a) (b)

Figure 6.6. U-strap FRP composite detail on State Street Bridge bent: (a) before protective coating was applied; (b) after protective coating was applied

CAPACITY VS DEMAND

The as-built and retrofitted bents were evaluated with respect to the following design earthquake levels: (1) 0.2g earthquake, or a 10% probability of exceedance in 15 years, which represented the design life of the retrofit, (2) 10% probability of exceedance in 50 years earthquake, and (3) 10% probability of exceedance in 250 years earthquake. A force reduction factor was used as (FHWA 1995):

(6.10)

$$R_F = 1 + 0.67(\mu - 1)\frac{T}{T_0} \le \mu$$

The elastic spectral displacement for an equivalent single degree of freedom system is:

$$S_d = \frac{S_a T^2}{4\pi^2} \qquad (6.11)$$

Assuming an "equal energy" approach, the inelastic displacement was obtained by multiplying the elastic displacement of Eq. (6.11) by:

$$R = \frac{\mu}{\sqrt{2\mu - 1}} \qquad (6.12)$$

Damage levels are outlined in what follows for both the as-built and retrofitted bent. These damage levels are associated with the level of nonlinearity required to achieve the inelastic displacement demand predicted at different hazard levels (i.e. ductility levels). This information was obtained from conducting pushover analyses.

As-built Bent

For the as-built bent, the period was T = 0.86 sec; the ductility was obtained from the model of Fig. 4.6 as μ = 2.9, which yields R_F = 2.27, and R = 1.32. For the three design earthquake levels, the following applies: (1) 0.2g earthquake, or a 10% probability of exceedance in 15 years: The peak spectral acceleration from the design spectra was 0.27g, with a lateral force demand F_d = 1,272 kN and reduced force demand F_r = 560 kN. The capacity as given in Fig. 6 was larger than the demand, and this design level is satisfied. The elastic displacement demand approximated with the first mode spectral displacement of Eq. (6.11) was 50mm, which when amplified by R was 66mm; this was less than the as-built bent capacity of 103mm, as shown in Fig. 4.6; (2) 10% in 50 years earthquake: The spectral acceleration was found from the design spectra as 0.41 g, with a lateral force demand F_d = 1,935kN and reduced force demand F_r = 852 kN. The capacity from Fig. 6 was higher than the demand and this design level is satisfied. The elastic displacement demand was 75mm, and when amplified was 99mm, which is less than the as-built bent capacity of 103mm; (3) 10% in 250 years earthquake: The peak spectral acceleration from the design spectra was 1.42g, with a lateral force demand F_d = 6,694 kN and reduced force demand F_r = 2,949 kN. Since the capacity from Fig. 4.6 was 1,352 kN, this design level is not satisfied. The elastic demand displacement was 261mm, which when amplified for inelastic effects was 344mm, and exceeded the capacity of the as-built bent of 103mm.

Retrofitted Bent

For the bent retrofitted with CFRP composites the period was T = 0.91 sec; the usable displacement ductility was μ = 5, R_F = 3.68, and R = 1.67. For the three design earthquake levels the following applies: (1) 0.2g earthquake or a 10% probability of exceedance in 15 years: The peak spectral acceleration was 0.26g with a lateral force demand F_d = 1,228 kN, and reduced force demand F_r = 334 kN. The capacity from Fig. 4.6 was larger so this design level is satisfied. The elastic demand displacement was 53mm, which when amplified was 89mm, and is less than the retrofitted bent capacity of 325mm, as shown in Fig. 4.6; (2) 10% in 50 years earthquake: The spectral acceleration was 0.39g, with a lateral force demand F_d = 1,837 kN and reduced force demand F_r = 499 kN. Since the capacity, from Fig. 4.6, was greater than 419 kN this design level is satisfied. The elastic demand displacement was 80mm, which when amplified was 134mm, which is less than 325mm; (3) 10% in 250 years earthquake: The peak spectral acceleration was 1.35g, with a lateral force demand F_d = 6,365 kN and reduced force demand F_r = 1,730 kN. Since the capacity from Fig. 4.6 was 1,864 kN this design level is satisfied. The elastic demand displacement was 278mm, which when amplified for inelastic effects became 464mm, which is more than 325mm. The true performance regarding the maximum inelastic displacement of the CFRP retrofitted bent is expected to be higher than that predicted by DRAIN-2DX. This is due to inability of the program to handle CFRP strap breakage, and column reinforcement anchorage loss, which were found to contribute to inelastic deformations beyond those predicted by DRAIN-2DX and before instability could be reached in in-situ tests of similarly retrofitted bridge bents (Pantelides et al. 2002b).

7. System Components

Contained in the special provisions for the project were the requirements for the materials to be used on the project. Mechanical properties for all materials were reviewed along with long-term durability data.

The material supplier had to exhibit a history of the chosen materials used on the project. This was to be done by submitting a list of past and current projects on which the materials have been used.

Special Provisions IM-80-3(126)123 “The composite wrap system shall have the following minimum initial properties as determined by ASTM D-3039, D-3171 and D-4065. These properties were used to develop the preliminary design thickness shown on the drawings but do not account for material property losses due to environmental aging. Note that actual construction thickness shall take into account environmental aging in accordance with long-term durability data”.

1 Minimum Initial Properties		
2 Properties at 22° C	**3 Min. Values**	**4 ASTM Test Method**
Ultimate Tensile Strength	960 N/mm^2	D3039
Tensile Modulus	73100 N/mm^2	D3039
Ultimate Elongation	1.3 %	D3039
Thickness/Layer	1 mm	
Primary Fiber Direction	0° Unidirectional	
Weight per m^2	870 g	
Strength per mm	0.972 kN/layer	D3039
Fiber Volume	40%	D3071
Glass Transition Temperature	60° C	D4065

Note: All tests are in the primary fiber direction and before impregnation with epoxy.

The directions for computing the actual design thickness was clearly spelled out in the Special Provisions IM-80-3(126)123.

Durability test data was also a major consideration for selection of the material. All materials submitted must meet the following durability test data.

5 Durability Test Environments

Environment	Conditioning Methods	Exposure Conditions	Test Duration
Water	ASTM D 2247 ASTM E 104	100% HUMIDITY AT 38 ° C ± 10° C	Initial, 1000, 3000, & 10,000 hours
Salt Water	ASTM D 1141 ASTM C 581	Immersion at 23° C ± 10° C	Initial, 1000, 3000, & 10,000 hours
$CaCO_3$ Solution	ASTM C 581	Immersion in CaCO3 at 23° C ± 10° C and pH 9.5	Initial, 1000, 3000, & 10,000 hours
Fuel resistance	ASTM C 581	Immersion in fuel at 23° C	Initial, 4 hours
Dry Heat	ASTM D 3045	50 ° C	Initial, 1000, 3000, & 10,000 hours
Freeze/Thaw	None	Cycle between 100% humidity at 38 ° C and freezing at –18 ° C	Initial, 20 cycles @ 24 hours/cycle

NOL ring tests were also a requirement for this project. The Naval Ordinance Lab (NOL) tests are performed on 508 mm inside diameter rings fabricated from the composite wrap system and shall meet the following minimum values:

6 Minimum Burst Strength per NOL Ring Test

Property	Value
Ultimate hoop stress	9600 N/mm^2
Hoop modulus	73100 N/mm^2

The other material of great importance on this project is the structural adhesive. The structural adhesive is not used on the round columns to bond the FRP to the concrete substrate. The cab beams however require a strong bond to the concrete substrate in order to properly transfer the load from the concrete to the FRP. Requirements for the structural adhesive are shown below.

7 Structural Adhesive Minimum Properties		
Properties	**Min. Values**	**ASTM Test Methods**
Tensile strength	24.8 Mpa	D-638
Elongation at brake	1 %	D-638
Modulus of elasticity 7 days	4.48×10^3 MPa 2.69×10^3 MPa	D-638
Flexural strength	46.8 Mpa	D-790
Shear Strength (14 days)	24.8 Mpa	D-732
Deflection temperature	47° C	D-648
Water absorption	0.03%	

Gerber Construction Corporation in conjunction with their material supplier Sika Concrete Restoration Systems were awarded the contract for the project with the following materials that met or exceeded the requirements stated above.

SikaWrap Hex 103C Composite Material

8 Laminate Properties					
Description	**Tensile Strength**	**Tensile Modulus**	**Elongation**	**Nominal Thickness**	**Tensile Strength per inch width**
UD Carbon fabric (wet lay-up)	139000 N/mm^2	73100 N.mm^2	1.33%	1 mm	24.7 kN

The adhesive chosen was Sikadur 31, Hi-Mod Gel, a 2-component, 100% solid, moisture-tolerant, high-modulus, high-strength, structural epoxy paste adhesive.

8. COLUMN AND CAP BEAM REPAIR

A total of five bridges were rehabilitated in the period of 2000-2001 on I-80 in Salt Lake City using the special provisions of the *Supplemental Specifications* (UDOT 1999). Four of the bridges had their columns strengthened with CFRP composites. A total of 73 columns were strengthened with CFRP composites with the same carbon fiber/epoxy resin system as the State Street Bridge with a wet-layup under ambient temperature curing conditions; the columns were circular with a diameter of 914 mm and the column height ranged from 7.00 to 7.93 m; the number of CFRP layers varied along the column height. Only the State Street Bridge was seismically retrofitted and the retrofit included four bridge bents as shown in Figs. 8.1-8.4. The goal of the seismic rehabilitation on the I-80 State Street Bridge was to improve the displacement ductility of the bridge. The design for the CFRP composite is given elsewhere (Pantelides et al. 2001b).

A carbon fiber/epoxy system was implemented using a wet-layup under ambient temperature curing conditions. The required number of layers is shown in Figs. 8.1 and 8.2 as lower case (n). Using the concepts of Strength Capacity and environmental durability strength reduction factor, the number of layers used in construction was determined as shown in capital letters (N) in the same figures. It should be noted that the thickness of one layer of CFRP composite used in the actual retrofit application was less than that assumed in the design, which increased the number of layers applied.

During construction, a saturating machine was used, which assured uniformity of the CFRP composite properties. In order to maintain a relatively constant fiber volume the following procedure was used: a small area of dry carbon fiber was weighed, and was then saturated through the saturator and weighed again. From previous testing of tensile coupons the optimum ratio of the two weight measurements was known; the opening of the saturator was then adjusted to produce the desired weight ratio. This was done at the beginning of every working day in order to minimize variations in the CFRP composite properties. Visual inspection of the bridge had revealed that there was limited delamination of the concrete cover at the bent cap. However, there was no evidence of electromechanical corrosion.

For a CFRP composite retrofit design to be successful, it is very important that the delaminated concrete be removed, and be replaced by shotcrete or equivalent material at the substrate to achieve a satisfactory force transfer from sound concrete to the CFRP overlays. It should be noted that before any application of CFRP composites, new concrete had to be cast as shown in Fig. 8.2, to form a suitable surface for the vertical overlay sheets going over the bent cap and onto the column to form the “U-strap”. The straps were brought down 305 mm below the bottom of the bent cap before they were clamped, to avoid stress concentration effects, as shown in Fig. 8.2. The gap left between the strap, the bent cap and the column was filled with structural foam.

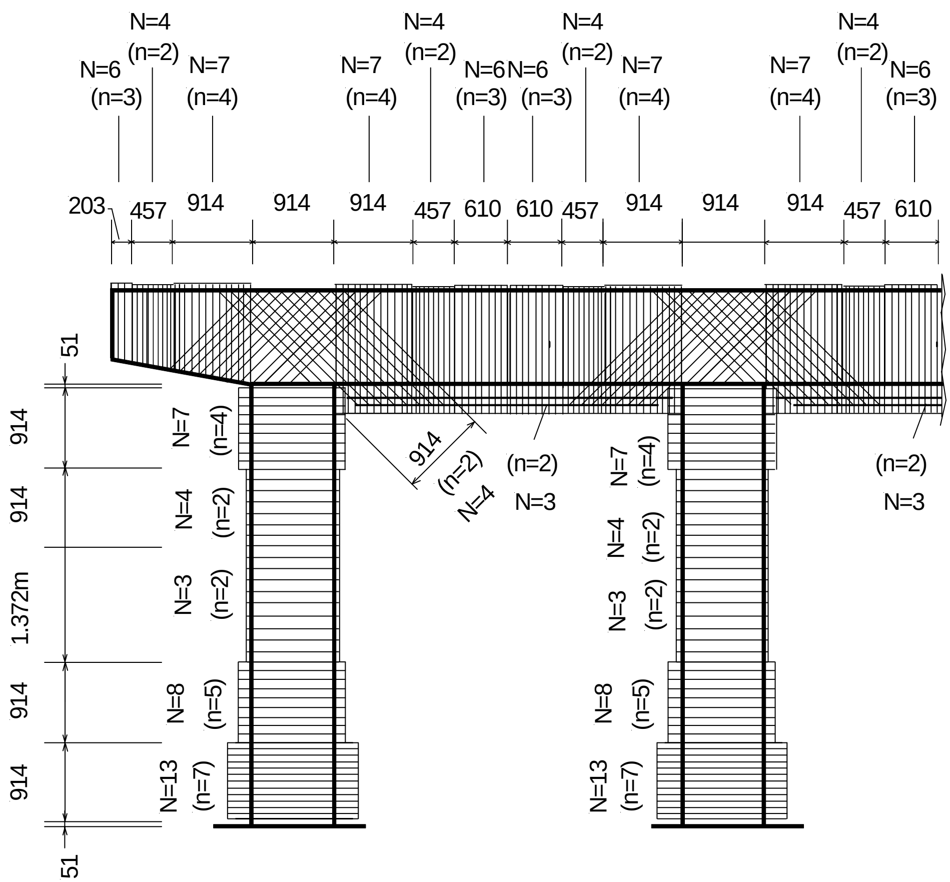


FIGURE 8.1. CFRP COMPOSITE DESIGN NUMBER OF LAYERS (N), AND CONSTRUCTED NUMBER OF LAYERS (N), FOR COLUMNS, BENT CAP, AND JOINT ANKLE WRAP.

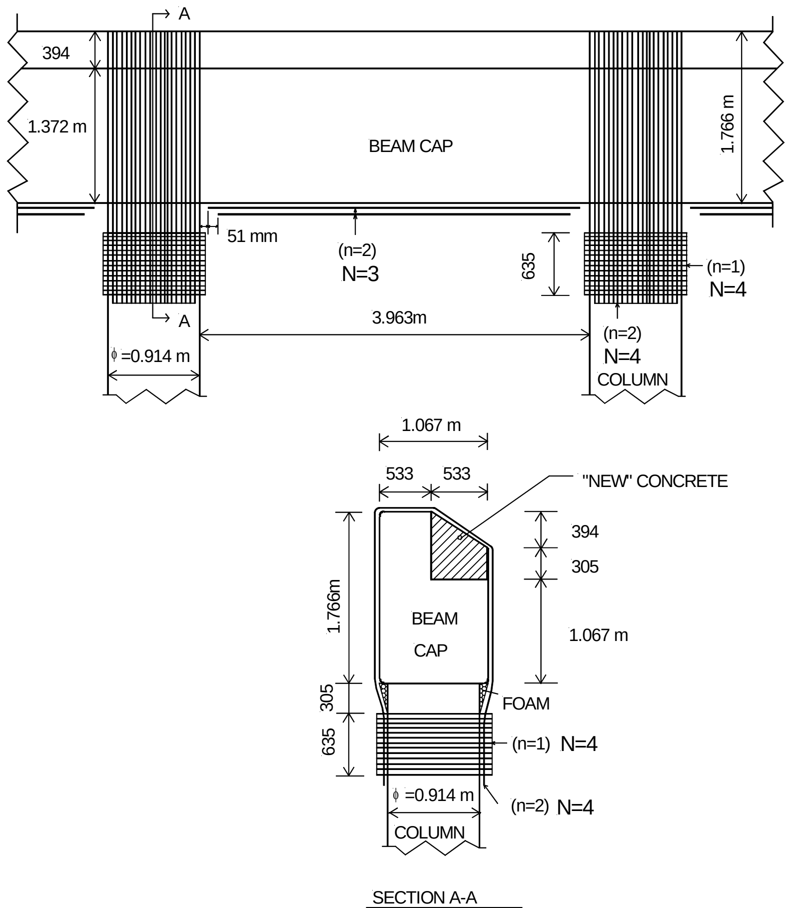


FIGURE 8.2. CFRP COMPOSITE DESIGN NUMBER OF LAYERS (N), AND CONSTRUCTED NUMBER OF LAYERS (N), FOR COLUMN TO BENT CAP "U-STRAP".

Figure 8.3. CFRP composite detail near footing.

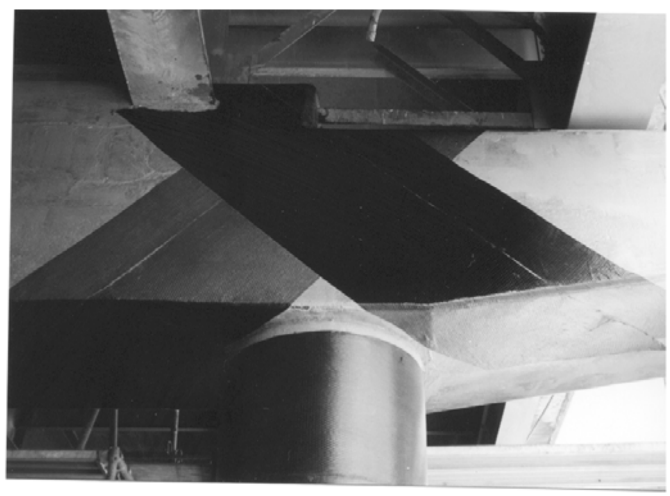


Figure 8.4. CFRP composite detail near bent cap-column joint.

The sequence of CFRP composite application was as follows: (1) the first layers were placed on the columns (N=3) as shown in Fig. 1; (2) the remaining layers were placed on the columns to complete the required number, N, starting at the column bottom and proceeding to the top; the CFRP composite was continued underneath the soil all the way to the top of the footing, as shown in Fig. 8.3, but was stopped short of the footing surface by 51 mm to avoid any strength and stiffness increase; (3) the flexural strengthening of the bent cap was accomplished by successively applying the layers at the bottom of the beam as shown in Figs. 8.1 and 8.2 (at the ends of the beam, near the columns the sheets were terminated 51 mm from the end of the previous sheet, to avoid stress concentrations from the retrofit as shown in Fig. 8.2); this is less than the ACI draft report recommendations for allowable termination points of 150 mm (ACI 2000); (4) the diagonal sheets were applied over the bent cap to column joints in the ankle wrap configuration at ±45 degrees from the horizontal, as shown in Figs. 8.1 and 8.4; (5) the four-sided wraps were then applied on the bent cap as shown in Fig. 8.1 at the various thicknesses, N, which varied from N=4 to N=7; (6) the "U-strap" vertical sheets were applied as shown in Fig. 8.2 over the bent cap and down to the column, and subsequently the circular clamping CFRP sheets were applied over the U-strap sheets as shown in Fig. 6.5; and (7) the protective coating was applied over the CFRP. A detail of the bent cap–column joint after the coating and structural foam were applied is shown in Fig. 6.6(b).

Two bents were overlaid with CFRP composite simultaneously. The total time required for the retrofit of all four bents with the CFRP composite was approximately three months. The four bridge bents required a total of 3,300 m^2 of carbon fiber fabric, 1,900 liters of epoxy resin, 400 liters of adhesive gel, and 500 liters of finish coating. Approximately 110 strain gages were installed both in interior layers as well as the exterior CFRP composite layer in both the columns and the bent cap for long-term health monitoring of the FRP composite, as shown in Fig. 8.3.

The FRP composite flat coupon tests during bridge-site construction showed that the FRP composite met the specifications; in addition, no other design deficiencies were found in terms of the FRP composite material. Remedial actions involved epoxy injection of voids; no other repairs were necessary such as additional FRP composite layers.

CONSTRUCTION REQUIREMENTS AND ISTALLATION

The contractor was required under the *Supplemental Specification* (UDOT 1999) to submit certain construction-related items in the submittal as follows:

History-of-Use Documentation

Requiring a history-of-use documentation is standard practice for most relatively new construction products. The *Supplemental Specifications* (UDOT 1999) required that the CFRP composite system shall have been in use for at least two years on related transportation projects. A history of such projects with material quantities, dates, and locations was required. During the present project, the contractor, the Utah DOT inspectors, the manufacturer's representative and the Quality Control and Quality Assurance inspectors were required to attend a training session in which the details of the *Supplemental Specifications* (UDOT 1999) were discussed and demonstrated to the extend possible on scaled-down specimens for hands-on experience and establishment of good practice.

Test Data for Strength and Modulus of Elasticity

Required test data for mean and standard deviation of the strength and modulus properties of the CFRP composite were to be for the same fabric type, areal weight, resin formulation, weight fraction, and cure conditions as those to be used in construction. The test data included 50 tensile coupons from plates as well as ten NOL rings.

Test Data on Environmental Aging

Environmental test data used in determining the environmental durability strength reduction factor (ϕ) by the statistical method were to be for the same fabric type, areal weight, resin formulation, weight fraction, and cure conditions to be used in construction.

Requirements for Detailed Thickness of Composite Calculations

The requirements for detailed thickness calculations considered the definition of the Strength Capacity as outlined previously. Details of fabric impregnation, application, curing, drawing details of column and bent cap CFRP overlays, materials, aspects of the repair procedure, material suppliers lists, product data sheets, material safety data sheets, storage and handling requirements, and certification of compliance for all materials were required. These requirements are believed to constitute the minimum for standard practice in FRP composite retrofit projects.

Listing of Quality Control (QC) Personnel

A list of QC personnel, their work history and their training was requested in the special provisions of the *Supplemental Specifications* (UDOT 1999); this is especially needed in construction of relatively new materials and processes. This is the first application of requiring such listing and resumes of QC personnel for an FRP composite bridge retrofit project.

INSTALLATION REQUIREMENTS

The special provisions of the *Supplemental Specifications* (UDOT 1999) require certain installation procedures for the CFRP composite column jackets, and different installation procedures for the CFRP composite bent cap overlays with respect to surface preparation.

Surface Preparation and Mandated Repairs

The requirement in the special provisions for the columns was that the surface should be free of fins, sharp edges and protrusions that could damage the fibers or cause voids or depressions behind the installed CFRP jacket. Surfaces should be cleaned and free of dust, grease or any other foreign matter. Depressions greater than 25mm in diameter by 2 mm deep were to be filled with structural adhesive. By contrast, the requirements for the bent cap were that the surfaces should be hydro-blasted with a minimum pressure of 276 MPa, at a rate of 189 ml/sec; a thickness not less than 2 mm of the concrete substrate should be removed by hydro-blasting. Prior to application of the CFRP composite, the bent cap surface was to be coated with a layer of structural adhesive of 2mm minimum thickness. The provisions for the surface preparation of the bent cap were more stringent than the columns, since the bond of the CFRP composite to the bent cap is critical for improved structural performance (Pantelides et al. 2001a).

Application Conditions

The special provisions specified that concrete surfaces should be dry at the time of installation of the CFRP composite, and the hydro-blasted surfaces should be dried thoroughly for a minimum of three days prior to the installation of the CFRP composite. The ambient air temperature was to be at least 5 ^{0}C, and the relative humidity no greater than 80% at the time of application of the CFRP composite. This provision necessitated the application of the CFRP composite in two summers of consecutive years due to cold weather conditions. The two east bents shown in Fig. 4.1 (one for the eastbound bridge and one for the westbound bridge) were retrofitted during the summer of 2000 and the two west bents in the summer of 2001.

Material Application

A total of 3,300 m^2 of a carbon fiber/epoxy resin composite system were used in the retrofit, which is the largest amount to be used in a bridge retrofit project in the U.S. The alignment, orientation, and fitting of the carbon fibers in the special provisions was to be in accordance with the detailed design drawings as submitted. Splices were to be staggered so that the minimum distance between splices would be 150mm. The horizontal orientation of the carbon fibers should not deviate from a horizontal line more than 15mm over a length of 300mm. This is similar to the allowable fiber misalignment of the ACI 440 draft document (ACI 2000).

Protection of Adjacent Surfaces

The special provisions specified that during application of the CFRP composite, the contractor must protect adjacent surfaces not receiving the material from contamination.

Curing

The special provisions specified that before applying a finish coat, the CFRP composite should be adequately cured. The degree of cure of the FRP composite was indicated by tests of core samples according to ASTM D4065 for Glass Transition Temperature (ASTM 2001). The work should be protected during the curing process from large temperature variations, and the temperature should remain within the temperature range, as recommended by the manufacturer. If a high-temperature cure were to be used, the contractor was required to maintain the temperature of the entire curing surface within 10 ^{0}C of that recommended by the manufacturer, and monitor the surface temperature hourly during the curing process.

Finish Coat

In the special provisions, the entire bridge bent was required to receive a finish coating. However, the coating was not to be applied until after curing was adequate, all QC/QA testing was performed, and any required repairs had been made.

Cooperation with Owner

The University of Utah in co-operation with the UDOT is currently performing long-term evaluation of the CFRP composite retrofit of State Street Bridge. In the special provisions, this was stated explicitly with the clarification that part of this evaluation would be occurring at the time of installation, which might entail attachment of sensors and wiring. The contractor was informed that he would have to cooperate during this installation.

10. Materials and Labor Analysis

9 Material Usage and Labor

The following analysis details the material usage and cost for the I-80 State Street Bridge project. The usage will be broken down per column and cap beam.

The retrofit program was conducted over two construction seasons. The east side of the bridge was done from August 21, 2000 through September 25, 2000. The west side of the structure was completed from June 4, 2001 through June 25, 2001.

Each side of the bridge consisted of two bent caps, each with 4 columns per bent cap. The information presented was compiled from average number in the daily production logs provided by the contractor.

SikaWrap Hex 103C Composite Material was the material chosen for this project. The laminate properties are listed below.

10 Laminate Properties

Description	**Tensile Strength**	**Tensile Modulus**	**Elongation**	**Nominal Thickness**	**Tensile Strength per inch width**
UD Carbon fabric (wet lay-up)	139000 N/mm^2	73100 $N.mm^2$	1.33%	1 mm	24.7 kN

The material physical properties are as follows:

The fabric weight is 18 oz/yd^2 or (610 g/m^2).

The formulation to convert the aerial fiber weight into rolls is:

Rolls = $ft^2/625$

In order to maintain the specified % resin content one gallon was used to impregnate 70 ft^2 of fabric.

The resin mixture is:

Part A 100% by weight

Part B 40% by weight

East Side

East Side Columns		
Column	Square Footage	Man Hours
9	1071.04	38.36
10	1071.04	38.36
11	1071.04	38.36
12	1071.04	38.36
13	1071.04	38.36
14	1071.04	38.36
15	1071.04	38.36
16	1071.04	38.36
Total	**8568.32**	**306.88**

East Side Bent Caps		
Bent	Square Feet	Man Hours
Bent 3	9108.83	556.6
Bent 4	9108.83	556.6
Total	**18217.66**	**1113.2**

West Side Columns		
Column	Square Feet	Man Hours
1	1074.28	49
2	1074.28	49
3	1074.28	49
4	1074.28	49
5	1074.28	49
6	1074.28	49
7	1074.28	49
8	1074.28	49
Total	**8594.24**	**392**

West Side Bent Caps		
Bent	Square Feet	Man Hours
Bent 1	9248.31	658.9
Bent 2	9248.31	658.9
Total	**18496.62**	**1317.8**

25 West Side

Labor Cost

West Side

West Side Columns				
Column	**Square Feet**	**Man Hours**	**Labor Rate**	**Labor Cost**
1	1074.28	49	$ 22.00	$ 1,078.00
2	1074.28	49	$ 22.00	$ 1,078.00
3	1074.28	49	$ 22.00	$ 1,078.00
4	1074.28	49	$ 22.00	$ 1,078.00
5	1074.28	49	$ 22.00	$ 1,078.00
6	1074.28	49	$ 22.00	$ 1,078.00
7	1074.28	49	$ 22.00	$ 1,078.00
8	1074.28	49	$ 22.00	$ 1,078.00
Total	**8594.24**	**392**	$ 22.00	$ 8,624.00

West Side Bent Caps				
Bent	**Square Feet**	**Man Hours**	**Labor Rate**	**Labor Cost**
Bent 1	9248.31	658.9	$ 22.00	$ 14,495.80
Bent 2	9248.31	658.9	$ 22.00	$ 14,495.80
Total	**18496.62**	**1317.8**	$ 22.00	$ 28,991.60

East Side

East Side Columns				
Column	**Square Footage**	**Man Hours**	**Labor Rate**	**Labor Cost**
9	1071.04	38.36	$ 22.00	$ 843.92
10	1071.04	38.36	$ 22.00	$ 843.92
11	1071.04	38.36	$ 22.00	$ 843.92
12	1071.04	38.36	$ 22.00	$ 843.92
13	1071.04	38.36	$ 22.00	$ 843.92
14	1071.04	38.36	$ 22.00	$ 843.92
15	1071.04	38.36	$ 22.00	$ 843.92
16	1071.04	38.36	$ 22.00	$ 843.92
Total	**8568.32**	**306.88**	$ 22.00	$ 6,751.36

East Side Bent Caps				
Bent	**Square Feet**	**Man Hours**	**Labor Rate**	**Labor Cost**
Bent 3	9108.83	556.6	$ 22.00	$ 12,245.20
Bent 4	9108.83	556.6	$ 22.00	$ 12,245.20
Total	**18217.66**	**1113.2**	$ 22.00	$ 24,490.40

Total	**53876.84**	**3129.88**		**$ 68,857.36**

Fiber	**6734.61**	Pounds
Resin	**3367.30**	Pounds (A + B)

26 Traffic Control

Traffic control on this project was critical. According to the special provisions the traffic on State Street had to be kept moving during the entire construction project. According to section 849.3.4 of the special provisions the lane restrictions were as follows:

A.M. Peek is 6:00 A.M. to 9:00 A.M.; west bound and north bound.
P.M. Peek is 4:00 P.M. to 7:00 P.M.; east bound and south bound.

Gerber Construction Inc. set “K” rail within the work area that did not encroach on either the east or west bound traffic lanes. These were kept in place during the entire project.

Between the hours stated above one lane was closed to traffic using cones in the off peak hours to permit the use of heavy equipment needed on the project. This equipment included all terrain forklifts with specially manufactured platforms, high-pressure water blasting equipment, and excavation equipment needed to expose the columns below the ground level.

26.1 Work Proceeding During Peak Hours – No Lane Closure

26.2 Work Zone During Off-Peak Hours

26.3 Removing Barriers At Completion of Project

Using conventional steel jacketing retrofit techniques the lane closure specified in the Special Provisions could not be achieved as written because a crane and other heavy equipment would need to be in the work zone for the duration of the project.

Also, if conventional methods were used the lanes would have been closed as stated above but the construction would have been done at night, which would have entailed moving k-rail twice a day thus increasing the project cost.

The use of FRP methods reduced the cost of the overall project by allowing the crew to make optimum use of the work zone during peek traffic hours.

10. INNOVATIONS

Standard specifications for externally applied FRP composites to concrete structures are in an evolutionary stage. The American Concrete Institute's Committee 440 report on FRP reinforcement for concrete structures (ACI 1996) includes information on FRP composite materials, properties and test methods, and design guidelines for external reinforcement. Currently, ACI Committee 440 is updating this report with information gathered from research (ACI 2000). In Europe, the EUROCRETE project has produced draft recommendations for utilization of non-ferrous reinforcement using modifications of existing design rules (Clarke et al. 1996). In 1997, the Japan Society of Civil Engineers published recommendations for design, testing and construction of concrete structures using continuous fiber reinforcing materials (JSCE 1997). The Canadian Standards Association, included a section in the Canadian Highway Bridge Design Code containing material properties and durability issues of FRP composites and Fiber Reinforced Concrete (FRC) for deck slabs, concrete beams, tendons, and barrier walls (Bakht et al. 2000). Recently, the International Conference of Building Officials Evaluation Service has published acceptance criteria for concrete strengthening using FRP composite systems (ICBO-AC125 2001).

The Interstate 80 State Street Bridge in Salt Lake City, was designed in 1965 according to the State of Utah Standard Specifications for Road and Bridge Construction, 1960. The concrete was designed with a compressive strength of 29 MPa and the steel with a yield stress of 280 MPa. The girders are welded steel plate beams. The CFRP composite design for seismic rehabilitation of the bridge is documented elsewhere (Pantelides et al. 2001b). This article presents the provisions developed for specification of CFRP composite materials, constructability issues related to the application of the CFRP composite, and quality control aspects of the implementation.

SPECIAL PROVISIONS FOR COLUMN AND BENT COMPOSITE WRAP

As part of the *Supplemental Specifications* of the construction contract (UDOT 1999), two special provisions were developed: (a) *Special Provision 525S* for column composite wrap, and (b) *Special Provision 526S* for bent composite wrap. The reason for the two provisions was that in addition to the State Street Bridge, four other bridges were to receive strengthening of their columns with CFRP composite jackets, whereas the State Street Bridge would receive a complete seismic strengthening with CFRP composites, which included the columns, bent cap, and bent cap column joints. This article presents the requirements of both special provisions of the *Supplemental Specifications* (UDOT 1999), developed prior to the ACI 440 draft document (ACI 2000) and the ICBO-AC125 document (2001).

Materials

E-glass fiber composites are used widely because of their lower cost. However, studies have shown possible durability problems in FRP bars made of E-glass in environments with high alkalinity such as in concrete (Katsuki and Uomoto 1995; Tannous 1997). An experimental study of cement-based specimens wrapped with FRP composite sheets, subjected to exposure of wet-dry cycling and freeze-thaw cycling was recently carried out (Toutanji and El-Korchi 1999); carbon fiber composite specimens showed no effects, whereas glass fiber composite specimens showed significant degradation in strength of the order of 10 % for freeze-thaw cycling, and 20% for wet-dry cycling. Due to the harsh weather conditions and the practice of using de-icing salts, the Utah Department of Transportation (UDOT) limited the materials selection to carbon fiber/epoxy resin composite systems as the only acceptable fiber reinforced composite materials for the rehabilitation. While the moisture and high alkaline environmental durability of both E-glass and carbon fiber composites are superior to that of steel, the carbon fiber composite is substantially more resistant than E-glass fiber composite. The selection of the carbon fiber/epoxy resin system was also based on the adequate history of successful application to concrete bridges in Utah and other States.

Considerable development and validation of the improved seismic performance of carbon fiber/epoxy resin materials has been conducted by the Defense Advanced Research Projects Agency (DARPA) through the Advanced Composites Technology Transfer/Bridge Infrastructure Renewal (ACCT/BIR) Consortium (Seible et al. 1995, 1997), and by the University of Utah on I-15 and other I-80 bridge bents (Gergely et al. 1998, Pantelides et al. 1999, 2001a). The choices between ambient and high-temperature cure, and between machine and hand wrapping techniques for carbon fiber/epoxy resin composites were options available to the contractor; however, precured casings were not an option. The high-temperature cure system with carbon FRP composites is a technique, which was used previously by the ACCT/BIR Consortium for seismic retrofit of columns (Seible et al. 1995, 1997). In 1996, a demonstration project at the I-80 Highland Drive Bridge in Salt Lake City used a high-temperature cure system with CFRP composites for the seismic retrofit of a bridge bent (Gergely et al. 1998).

The composite wrap system was required to meet minimum initial properties, as shown in Table 12.1. Other requirements included determination of jacket thickness, environmental durability rating, Naval Ordinance Laboratory (NOL) ring strength, and history of use. The NOL ring test started out as a 146 mm diameter ring test for strength and modulus determination of filament wound materials. The ring fabrication is performed according to ASTM D2291, and testing according to ASTM D2290 (ASTM 2001). A larger, 508 mm diameter ring, tested by internal hydraulic pressure has since replaced the smaller ring. In Table 12.1, the baseline properties were selected from the lower of supplier advertised properties of carbon fiber/epoxy resin material systems to permit a wide range of bid responses. The special provisions of the *Supplemental Specifications* (UDOT 1999) set forth the requirements for verification of proposed material properties and property retention under a prescribed set of environmental conditions. This set of conditions, of both exposure type and exposure time durations had been largely developed by the Defense Advanced Research Projects Agency under the Advanced Composites Technology Transfer/ Bridge Infrastructure Renewal program (Seible et al. 1995). With respect to the glass transition temperature (T_g), the requirement of a minimum T_g of 60 ^{0}C is based on a maximum expected service temperature of 42 ^{0}C. From actual tests after the CFRP composite application, the mean value of T_g was found to be 71 ^{0}C. It should also be noted that the value of T_g equal to 60 ^{0}C is also used in the ICBO-AC125 document (ICBO 2001).

Table 12.2. Durability Environment Parameters

Environment (1)	Conditioning Method (2)	Exposure Conditions (3)	Test Duration (4)
Water	ASTM D 2247 ASTM E 104	100%Humidity $38^0C\pm10^0C$	Initial, 1000, 3000 & 10000 hrs
Salt Water	ASTM D 1141 ASTM C 581	Immersion at $23^0C\pm10^0C$	Initial, 1000, 3000 & 10000 hrs
$CACO_3$ Solution	ASTM C 581	Immersion in $CACO_3$ at $23^0C\pm10^0C$ and pH 9.5	Initial, 1000, 3000 & 10000 hrs
Fuel Resistance Test	ASTM C 581	Immersion in Diesel Fuel at 23^0C	Initial, 4 hrs
Dry Heat	ASTM D 3045	50^0C	Initial, 1000, 3000 & 10000 hrs
Freeze/Thaw	None	Cycle between 100% humidity at 38^0C and freezing at -18^0C	Initial, 20 cycles @24 hours/cycle

In primary fiber direction and normalized to 40% fiber volume

Strength Capacity

The utilization of "Strength Capacity" as the structural strength requirement for column and bent cap CFRP overlays was used in the special provisions of the I-80 *Supplemental Specifications* (UDOT 1999) for the first time in highway bridge retrofit and rehabilitation. Strength Capacity is force per unit width or the product of the stress (S) and jacket thickness (T). The material allowable stress is defined as the mean ultimate strength, minus two standard deviations, (2σ). In addition, the ($Mean\text{-}2\sigma$) strength is multiplied by an environmental durability strength reduction factor, (ϕ), which has a value less than one. The *Supplemental Specifications* (UDOT 1999) required testing of 50 samples for the purpose of determining the mean and standard deviation strength values. The specimens were flat and straight and their length, width, thickness, gripping method and test procedure was according to ASTM D3039 (ASTM 2001). The allowable strength, S_a, is given as:

$$S_a = \phi \ (S_{mean} - 2\sigma)$$

(10.1)

The thickness of the CFRP composite to be constructed (T_c) is related to the design strength, S_d, and design thickness, T_d, using the definition of Strength Capacity as:

$$T_c = \left(\frac{S_d}{S_a} \right) T_d$$

(10.2)

Equations (10.1) and (10.2) allow different manufacturers with different systems to proportion their designs according to the properties of their material.

Environmental Durability Strength Reduction Factor (ϕ)

The utilization of an environmental durability strength reduction factor to account for environmental degradation of the FRP composite material was used in the ACTT/BIR program for the first time in bridge retrofit and rehabilitation (Seible et al. 1995). This factor was determined for any candidate laminate material by a log-log regression analysis, from periodic sample test results by projecting property data beyond the limitation of fixed-term test values. The durability test data required for determining ϕ in the *Supplemental Specifications* (UDOT 1999) are given in Table 12.2, and the expression for obtaining its value is given as:

$$\phi = \frac{Ultimate\ \ Strength\ \ (\mathrm{Pr}ojected\ \ to\ \ Life\ \ \mathrm{Re}quired\ \ Duration)}{Ultimate\ \ Strength\ \ (Initial\ \ Value)}$$

(10.3)

The *Supplemental Specifications* (UDOT 1999) specify that in the event insufficient data exist for determining a ϕ factor from Table 12.2, a prescribed value could be used. This prescribed value was: (a) ϕ=0.75 for a carbon FRP composite ambient temperature curing system, and (b) ϕ=0.80 for a high temperature curing system.

Minimum Burst Strength requirement of Naval Ordinance Laboratory (NOL) Rings

The minimum burst strength of NOL rings was set forth in the *Supplemental Specifications* (1999) with values for ultimate hoop stress of 960 N/mm^2, and for a hoop modulus of 73,100 N/mm^2. The above strength and modulus values were based on gross section, as normalized to 40 percent fiber volume. Cylinders for the excising of test rings were required to be made during each day of bridge-site CFRP jacketing or CFRP overlaying. The minimum burst properties described above for NOL rings are the same values as the flat laminate minimum design properties of the CFRP composite system, shown in Table 10.1. The ring test results are especially useful in determining exact properties of cylindrical shapes made by the same process used in FRP jacket fabrication, such as the case here for the State Street Bridge with circular columns.

Structural Adhesive

Minimum structural adhesive properties were required by the special provisions; these properties were modeled after well-known products, which had been used extensively in previous tests both in the laboratory and during in-situ simulated seismic testing of bridge bents (Pantelides et al. 1999, 2001a). The structural adhesives are high modulus, high strength, structural, epoxy paste adhesives that have been used in many building and bridge repair projects. The minimum adhesive properties are shown in Table 10.3. It should be noted that a deflection temperature of 47 ^{0}C is adequate under the present local conditions, where 90 percent of the bridge is in a shaded area and the maximum expected service temperature is 42 ^{0}C.

Table 12.3. Structural Adhesive Minimum Properties

Properties (1)	Minimum/Maximum Values (2)	Test Method (3)
Tensile Strength	24.8 N/mm^2 (min.)	ASTM D 638
Minimum Elongation at Break	1% (min.)	ASTM D 638
Modulus of Elasticity	4480 N/mm^2 (min.)	ASTM D 638
Flexural Strength	46.8 N/mm^2 (min.)	ASTM D 790
Shear Strength (14 days)	24.8 N/mm^2 (min.)	ASTM D 732
Deflection Temperature	47 ^{0}C (min.)	ASTM D 648
Water Absorption	0.03% (max.)	ASTM D570

Finish Coat

The finish coat material properties, drying times, and developed hardness were specified for an aliphatic urethane water-borne coating, which is graffiti resistant, abrasion resistant, stain resistant, and non-flammable. The finish coat provides excellent ultraviolet resistance and has rapid hardness development. The material based on which the specifications were written has been used as a finish coat on many bridge retrofit and rehabilitation projects as a coating for FRP composite jackets and for coating concrete and masonry. The properties of the coating specified for the seismic retrofit of State Street Bridge are given in Table 12.4. The drying requirements for the finish coat material are given in Table 12.5. The Konig/Sward index is a thickness measurement of drying paint.

Certificate of Compliance

A Certificate of Compliance with the requirements of the special provisions was specified prior to the use of all materials. A copy of the Certificate of Compliance was to be included in the Daily Construction Log for Quality Control/Quality Assurance (QC/QA) purposes. The fact that the material was used on the basis of the Certificate of Compliance did not relieve the contractor of any of the requirements of the plans and specifications.

Table 12.4. Finish Coat Material Properties

Property (1)	Requirement (2)
Color	Concrete Hue
Texture	Gloss
Density (Specific Gravity)	1.02 ± 0.06 g/cm^3
Solids content by volume	30%
Volatile organic compounds	0.06 g/cm^3

Table 12.5. Drying Requirements for Finish Coat Material

Dryness (1)	Dry Time (minutes) (2)	Hardness Development (3)	Konig/Sward Index (4)
Set to touch	25	At 2 hours At 4 hours	20/16 32/20
Dry to touch	55	At 8 hours At 24 hours	39/24 56/36
Through Dry	120	At 48 hours At 1 week	84/46 103/50

11. QUALITY CONTROL/QUALITY ASSURANCE PROGRAM

The *Supplemental Specifications* (UDOT 1999) distinguished between Quality Control and Quality Assurance. Specifying the appropriate responsibilities of each is important in FRP composite bridge retrofit and rehabilitation. Quality Control was the responsibility of the contractor, and required continuous monitoring. Quality Assurance was the responsibility of the project engineer. The following items were required:

Quality Control (QC) and Quality Assurance (QA) Inspectors

QC inspectors provided continuous monitoring of the work under the special provisions. They had full stop-work authority based on quality and technical merit. QA inspectors made periodic visits to the site and determined whether repairs were needed, and inspected the remedial actions.

Daily Construction Log

The contractor maintained a daily log, to be submitted to the project engineer, which included information on the structure location and number; the date and the name of the contractor's shift supervisor, names of crew members and contractor's QC inspector; materials traceability and process records; fabrication, installation, and inspection data to prepare as-built documents at the end of construction; material preparation and placement sequence, the number of layers, and total thickness measurements; ambient air temperature and humidity readings at the beginning and end of each shift; sequence of curing operations and surface temperature monitoring data; all material certificates, laboratory test results, on-site test results, quality control observations, and significant directives for remedial action.

Manufacturer's Representative

Under the special provisions, the contractor was required to procure the services of a representative of the FRP composite material supplier to inspect the surface preparation and witness application and curing of the material. At completion of the installation, the manufacturer's representative provided the project engineer with a Certificate of Compliance that the installation, including the surface preparation and curing, was performed in accordance with the supplier's recommendations. Other similar projects have allowed this activity but have not required it.

Finish Coat Inspection

The special provisions required a wet thickness gage measurement of coating. This measurement is important to insure long-term durability of the finish and is used on most structural coating applications.

Sampling and Testing

Under the special provisions, the contractor manufactured two types of FRP composite samples daily: (a) six flat panels 300mm x 300mm x 2 layers thick, and (b) three cylinders of 508mm inside diameter, 610mm tall and a thickness of five layers. The flat panels were used in producing tensile coupons and the cylinders were used in the production of NOL rings. The testing for the coupons was performed at an independent laboratory while construction was on- going. The NOL rings were stored for future long-term research. With the exception of the on-site NOL ring cylinders, all other specimens have been used for the same purpose in other FRP composite retrofit projects. However, the details of the special provisions of the *Supplemental Specifications (UDOT 1999)* prescribed a highly organized means of carrying out the sample preparation, collection, storage, identification, and testing of the samples by an independent testing laboratory.

In addition, core samples were collected randomly by the independent testing laboratory, which were 13mm in diameter and extended to the thickness of the CFRP composite material applied, for determination of the CFRP composite's thickness, glass transition temperature according to ASTM D4065, and the fiber volume according to ASTM D3171 (ASTM 2001); fifteen such samples were taken from State Street Bridge. The holes created by this sampling were filled with a highly filled epoxy.

Remedial Actions and Repairs

Upon notification by the project engineer regarding rejection of any portion of the work, the contractor was required to take remedial action to correct the cause of the rejection. The contractor was responsible for the cost of the remedial action and for coordinating the performance of the remedial action with the overall project schedule.

Repair procedures were performed according to the manufacturer's recommendations and the *Supplemental Specifications* (UDOT 1999). All repairs were subjected to the same application, curing and quality control provisions of the original work. All defects including bubbles, delaminations and fabric tears covering more than 15 mm x 15 mm (225 mm^2) of the surface area were repaired. The types of repairs allowed were: (1) injection with a compatible epoxy in such a way as not to trap air in the void area; this method when used alone was applicable for small bubbles and for delaminations less than 150 mm in diameter (17,700 mm^2); (2) application of additional layers of FRP composite; this method when used alone was applicable to tears and to deficiencies found during testing; the number of layers to be added and the overlap was approved by the project engineer; and (3) combination of epoxy injection and application of additional layers; this method was applicable to large bubbles, voids and delaminations greater than 150 mm in diameter (17,700 mm^2). The ACI 440 draft report (ACI 2000) contains similar default limits and repair method requirements.

Design Deficiencies

The *Supplemental Specifications* (UDOT 1999) prescribed that in the event that concurrent field installation testing determines that the properties of any given material lot are less than the values in the contractor's submittal data, the contractor shall retest and/or submit a design revision for review and approval by the project engineer. The adequacy of material properties was determined as follows: The special provisions specified that if the thickness, glass transition temperature, tensile strength from coupons, or fiber volume of three of the five samples from each site fell below prescribed values, assumed in the design by the contractor, another set of five samples should be tested; if three of the additional five samples fell below the prescribed values this constituted failure of the CFRP composite material at that bridge site and was cause for rejection by the project engineer. The submittal included engineering calculations and drawings to justify the number of additional carbon composite wrap layers to be applied, the orientation of the additional layers and the overlap length.

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12. LONG TERM DURABILITY

Goal of long term durability program

The reconstruction of the Interstate 15 corridor provided a unique opportunity to test bridge structures in-situ before demolition and replacement with the new bridges. These tests were carried out in 1998 (Northbound lanes) and 2000 (Southbound lanes), at the South Temple Bridge on Interstate 15. Simulated seismic load tests of reinforced concrete bridges with typical pre-1960 design details were carried out in 1998 and 2000. The in-situ tests included bridge bents retrofitted with carbon fiber reinforced polymer (CFRP) composites and bents in the as-built condition. The results of these tests were used to develop design guidelines for seismic retrofit of reinforced concrete bridges with typical pre-1960 design details. These research efforts developed the expertise required in the construction of the seismic retrofit with CFRP composite materials. In addition, they were helpful in the development of specifications for CFRP composite column jackets and composite bent wraps. The State Street Bridge on Interstate 80 was selected as the bridge where this CFRP composite technology would be implemented. Although other seismic retrofit schemes were also considered, such as steel jackets, the choice of the CFRP composite seismic retrofit was encouraged by the Federal Highway Administration as an application of innovative technology. The seismic retrofit was carried out in the summer of 2000 and 2001. The purpose of the durability program is to determine through non-destructive testing the long-term durability of the CFRP composite seismic retrofit of State Street Bridge on Interstate 80. Parallel studies are also being carried out currently at the University of Utah, funded by the U.S. National Science Foundation, to assist in the evaluation of durability of CFRP composites in the infrastructure through destructive testing.

INSTRUMENTATION

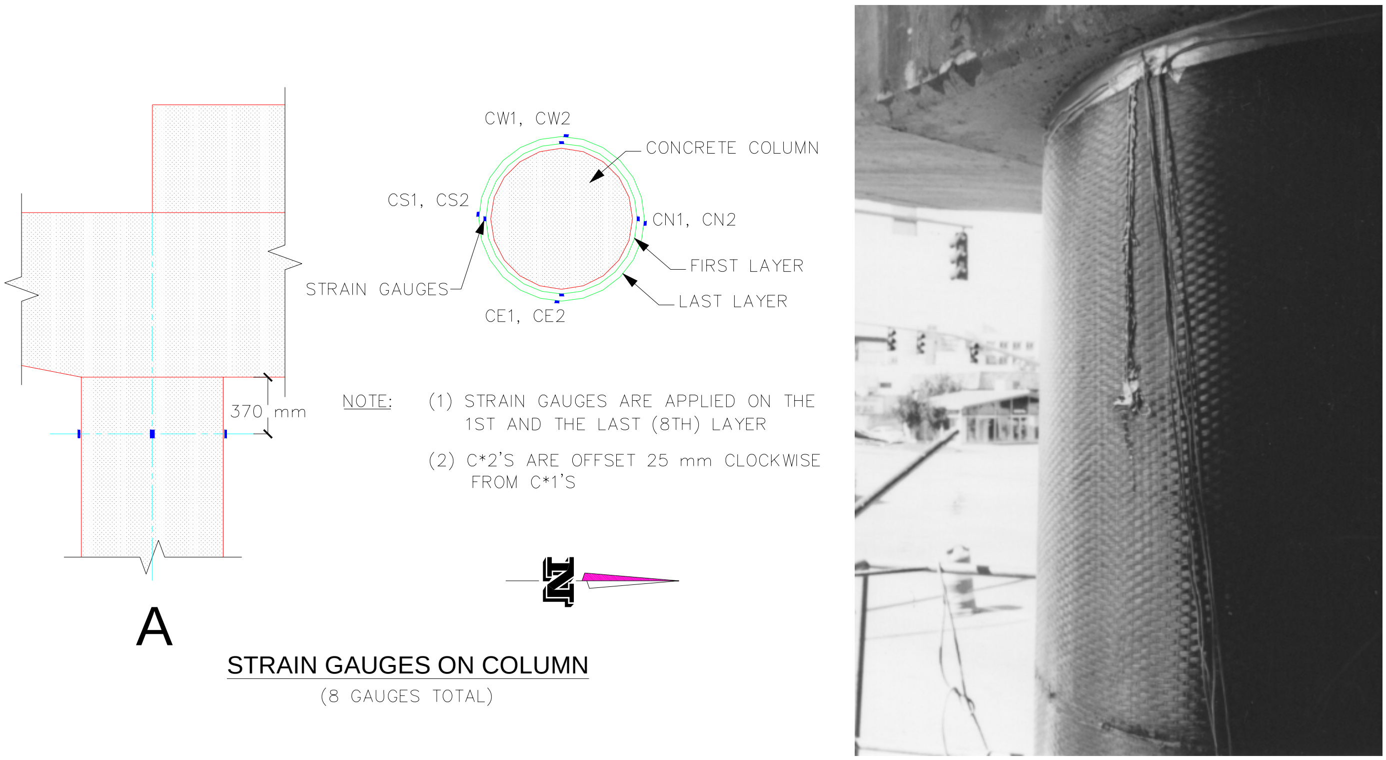
In order to evaluate the long term effects of the environment on the CFRP composite retrofit regarding the strength and structural characteristics of the State Street reinforced concrete bridge, it was instrumented with various types of sensors. The type of data that would be needed to properly evaluate these effects guided the choice of sensors to be used. Critical locations at the bridge bents, where the retrieved data would be most meaningful, were chosen, and the specific location for each sensor was determined. The total work plan for the instrumentation of the bridge involved applying 100 strain gauges (both embedded and on the surface), four thermocouples, two relative humidity/temperature sensors, and six tilt meters. Each sensor would be connected to an automated data acquisition system, which would take readings from each sensor at a specified time interval. A wireless Ethernet connection from the University of Utah to the data acquisition system would allow the collected data to be downloaded. The installation of these sensors and instruments occurred both during and after the application of the CFRP composite to the State Street Bridge bents. The following describes how each sensor was applied to the bridge, the location of each sensor, and when the application was performed.

Strain Gauges

The use of strain gauges is a common method of obtaining non-destructive data for the strength and structural characteristics of a structure. For this application, the long-term change in strain at specific locations provides useful data for the determination of strength degradation. Therefore, it was decided to apply strain gauges to all critical locations of the bridge bents in order to monitor the degradation where it would be most prominent. These locations include the joint of the column and the beam cap, the bottom of the midspan of the beam cap between two columns, the CFRP composite U-strap connecting the column to the beam cap, and the top and bottom of the column. The CFRP composite layer in direct contact with the concrete is expected to see the highest magnitude in strain, so strain gages were applied to both the first and last layers of CFRP composite in each location, so that the distribution of strain through the thickness of the composite could be obtained.

The method of applying each strain gauge to the CFRP composite is as follows: The precise location of each gauge is determined (each gauge is attached to one fiber with no overlapping in order to obtain the most accurate data). The area is sanded so that the gauge is attached to a smooth, fibrous surface. The area is then cleaned using a water-based acidic surface cleaner/conditioner, and then a water-based alkaline surface cleaner/neutralizer. Once the surface is dry, the strain gauge is attached to the CFRP composite using an adhesive specified by the manufacturer. Strain gauge wire is soldered to the gauges, and the connection is verified using a digital multimeter. Once the adhesion of the gauge to the CFRP composite and the wire connection is confirmed, a protective coating is applied over the strain gauge and the wire leads. The gauge is now ready to be connected to the data acquisition system and start taking measurements.

The application of strain gauges occurred throughout the construction phase of the project. Construction on the southeast bent of the State Street Bridge began in August 2000. The columns were the first portions of the bent to be wrapped. After the first CFRP composite layer was applied, at least one day (depending on temperature and humidity) was required to allow the epoxy resin to cure. Once it had sufficiently hardened, the first layer of strain gauges was applied to the top of the column in four locations as shown in Figure 12.1.



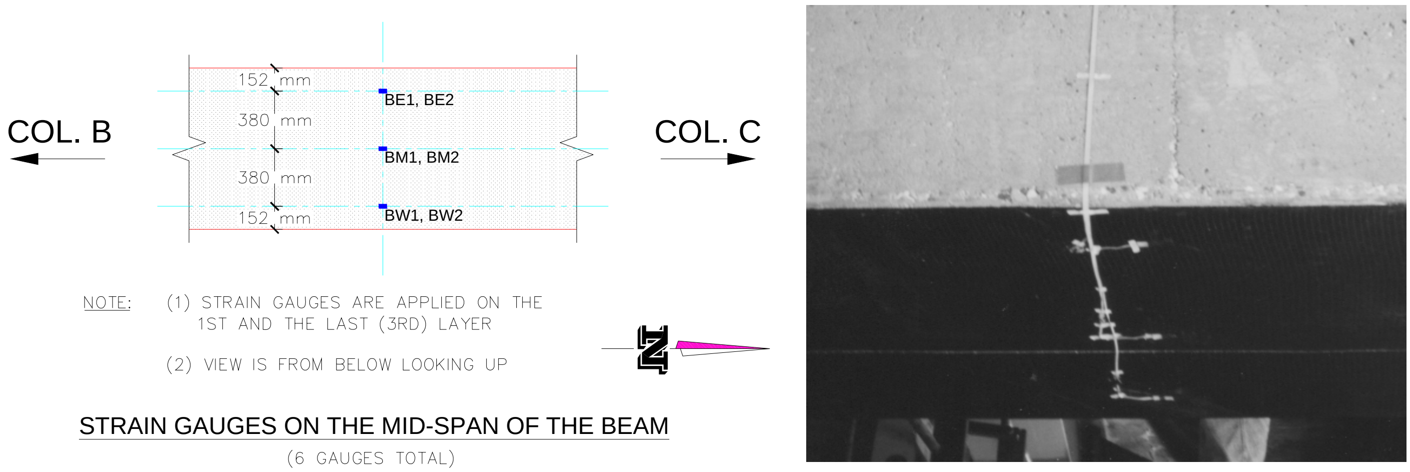
(a) **(b)**

Figure 12.1. Location of strain gauges on southeast column: (a) detail drawing; (b) actual application

The columns were then wrapped with the specified number of layers, and strain gauges were applied to the final layer at the locations shown previously.

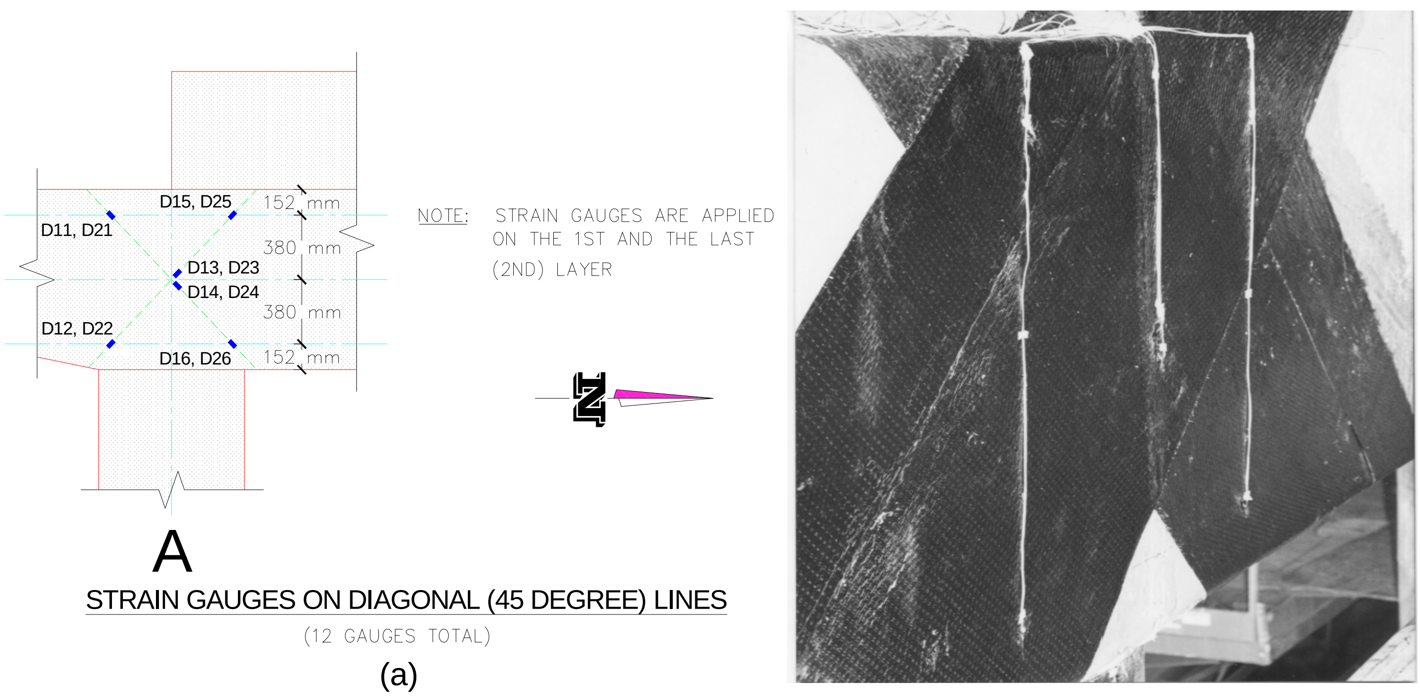
After the columns had been wrapped, CFRP composite was applied to the bottom of the beam cap. Strain gauges were applied at the mid-span of the beam cap in three locations as shown in Figure 12.2. Again, the gauges were applied on the first and last layers of the composite.

The bent cap-column joints were the third elements of the bent to be wrapped with CFRP composite. This was done by applying diagonal sheets, at ±45 degrees from the horizontal, to the joint area. The strength of the CFRP composite is in tension in the fiber direction. Therefore, the strain gauges were also applied at ±45 degrees, following the fiber direction. The locations and orientation of the bent cap-column joint strain gauges are shown in Figure 12.3.



(a) (b)

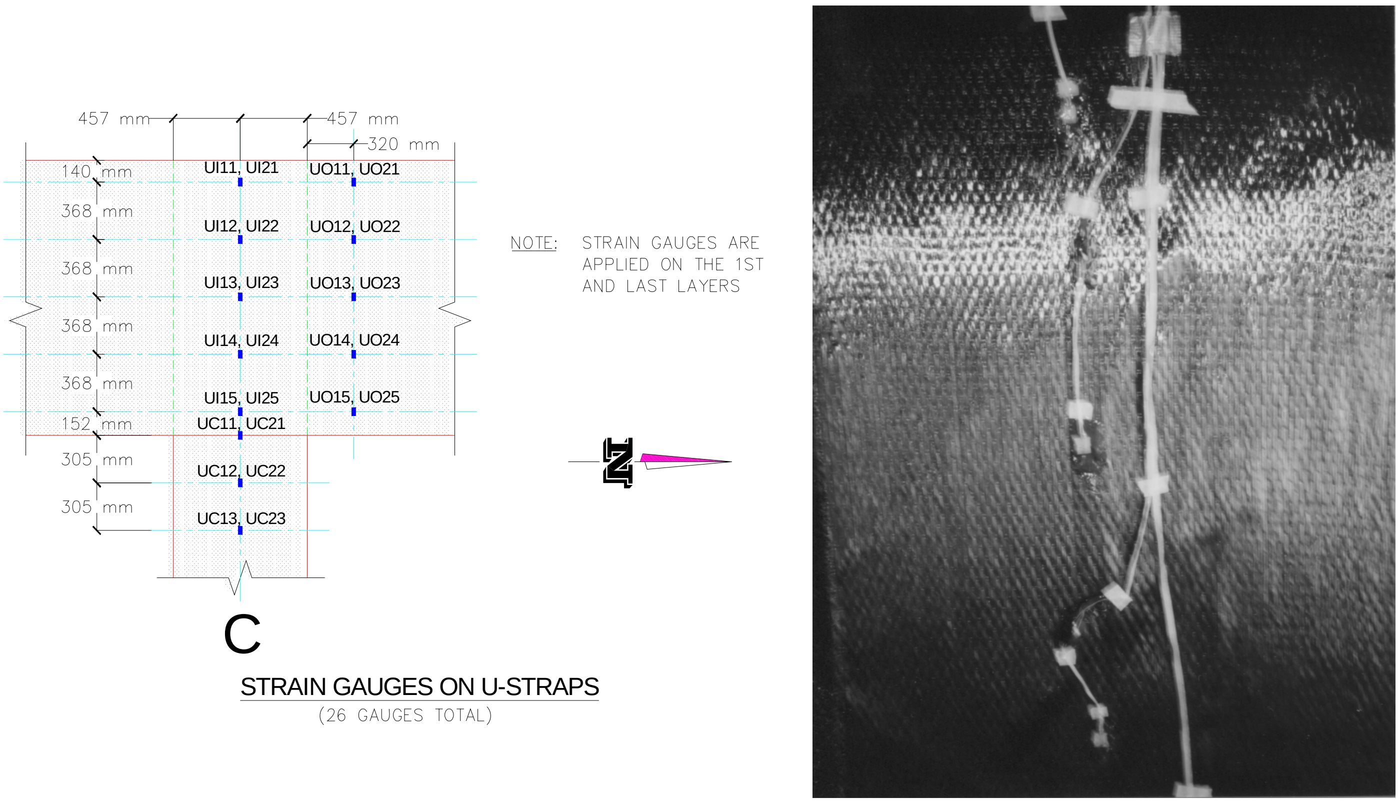
**Figure 12.2. Location of strain gauges on underside of beam cap: (a) detail drawing;
(b) actual application**



(b)

**Figure 12.3. Location of strain gauges on bent cap-column joint: (a) detail drawing;
(b) actual application**

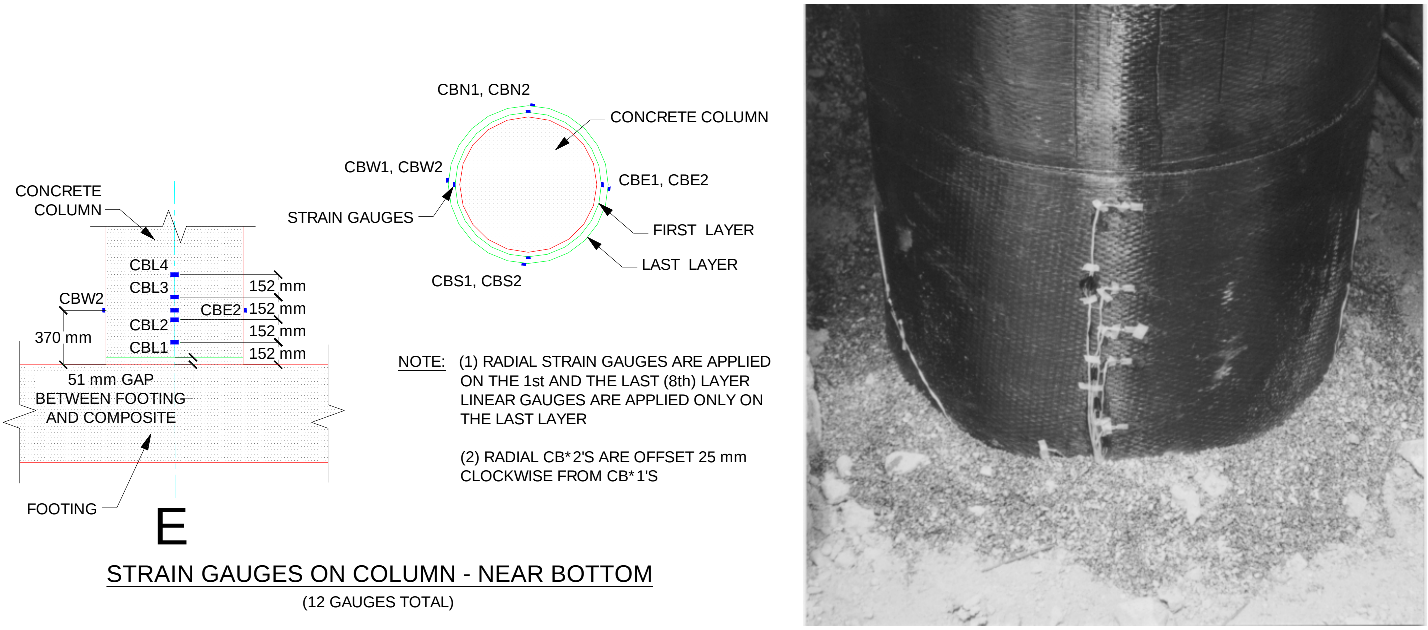
Finally, CFRP composite completely wrapped 4-sided stirrups were added to all four sides of the beam cap; at the location where the column joins the cap, a CFRP U-strap was placed over the beam and attached to the column. Strain gauges were placed in several locations down the U-strap and completely wrapped 4-sided stirrup, as shown in Figure 14.4. The gauges were applied vertically to follow the unidirectional fiber direction, and were applied to the first and last layers, as they were at all other locations. Application of the CFRP composite and all strain gauges to the southeast bent was completed in October 2000.



(a) (b)

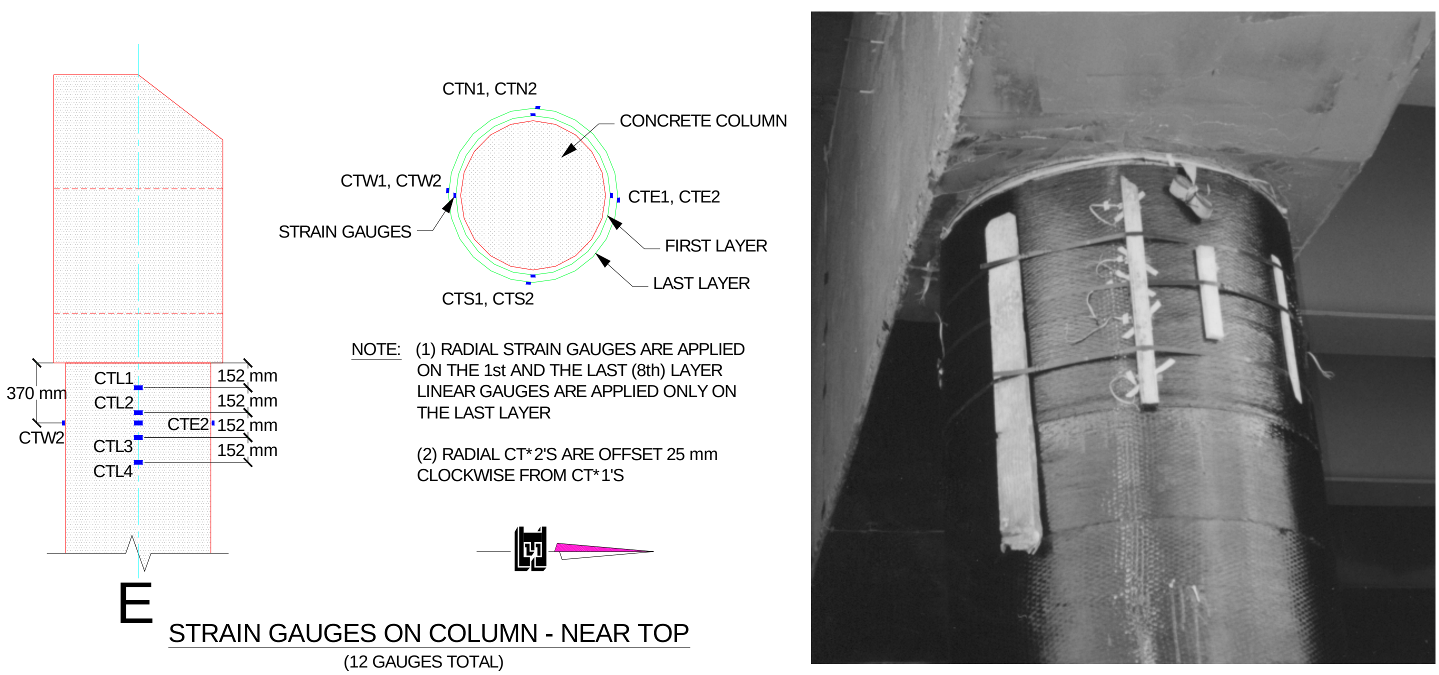
Figure 12.4. Location of strain gauges on U-strap: (a) detail drawing; (b) actual application

Construction on the southwest bent of State Street Bridge began in June 2001. The Work Plan called for strain gauges to be installed in the same locations on the west bent as on the east bent. This would provide multiple points for comparison, enabling greater accuracy when forming conclusions. For the west side, the Work Plan was modified to include strain gauges to be applied in locations not instrumented on the southeast bent. For instance, strain gauges were applied at the bottom of the southwest column as well as on the top, providing an opportunity to monitor the load/stress transfer through the column, underneath the soil surface, to the footing. In addition to gauges applied in the hoop direction, four linear strain gauges were placed on the last layer only, at 150 mm spacing. These gauges are intended to give information about the stress/strain profile along the length of the column. Linear strain gauges were also applied at the top of the column. The locations of these strain gages are shown in Figures 14.5 and 14.6.



(a) **(b)**

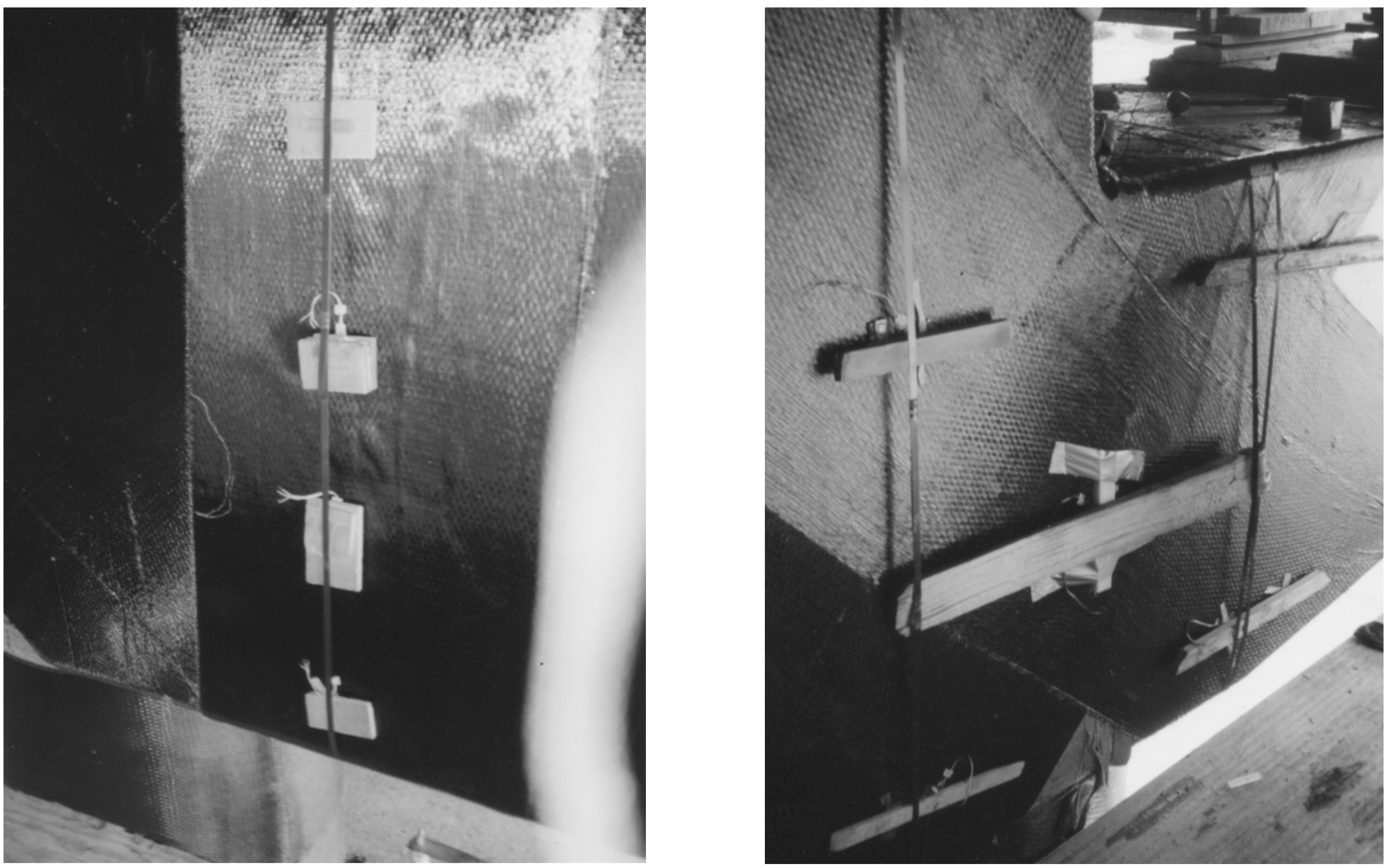
Figure 12.5. Location of strain gauges on southwest column bottom: (a) detail drawing; (b) actual application



(a) (b)

Figure 12.6. Location of strain gauges on southwest column top: (a) detail drawing; (b) actual application

A different adhesive was used for attaching the strain gauges on the west bent than on the east. It was selected because of its ability to withstand environmental effects such as large temperature and moisture variations. This adhesive, however, required that a constant pressure of at least 10 psi (69 kPa) be placed on the strain gauge for five hours. Therefore, banding steel straps were used to apply the required pressure for the amount of time necessary. An example of this is shown in Figures 12.6 and 12.7. Application of the CFRP composite and all strain gauges to the southwest bent was completed in July 2001. Where the strain gauge wire is covered by the composite, a thin, flat wire is used. Where the wire is exposed to the environment, a thicker, sheathed wire was spliced to the thin wire to provide more protection.



(a) (b)

Figure 12.7. Steel straps used to apply pressure to strain gauges as adhesive sets: (a) U-strap strain gauges; (b) bent cap-column joint strain gauges

Thermocouples

Thermocouples are a simple and common way of measuring temperature changes. Two wires of dissimilar metals are joined at one end, and a current is induced through the loop. As the temperature changes, the resistance in the loop will change. The change in resistance is linearly proportional to the change in temperature. Therefore, by using a thermistor as a reference, and knowing the type of metals in the thermocouple, the temperature at the end of the thermocouple (where the two wires are joined) can be determined. Thermocouples are useful because they are durable and can be used to determine air temperature, or they can be embedded into a material to find the internal temperature. In this application, it is important to know the temperature at all times because the CFRP composite is affected by even small temperature changes. It was decided that since other sensors would be monitoring the ambient air temperature, the thermocouples would be embedded into the concrete. That way, the temperature change in the concrete and through the thickness of the composite could be monitored. Four thermocouples were embedded into the southeast bent: two on the south end, and two on the north end. The locations of the thermocouples are shown in Figures 12.8 and 12.9. They are labeled S1, S2, N1, and N2. Small holes were drilled into the concrete, and the thermocouples were inserted into the holes. The remaining void in the holes was filled with an epoxy.

Relative Humidity/Temperature Sensors

Very little is currently known about the effect that moisture and humidity has on CFRP composites applied in field conditions. However, since it is known to affect the durability of reinforced concrete structures, it was determined that humidity would be an important variable to include in the analysis of the composite retrofit. Therefore, two Relative Humidity/Temperature Sensors were installed on the southeast bridge bent. They were mounted near the locations of the thermocouples, so the air temperature readings they provided could be compared to the internal temperature readings from the thermocouples, and a direct correlation between the readings could be established. See Figures 12.8 and 12.9 for sensor locations. The manufacturer did not provide any sort of mounting device, so a unique mounting system was devised. The RH/T Sensor was attached to a blue electrical box, which was then anchored to the concrete, as shown in Figure 12.9.

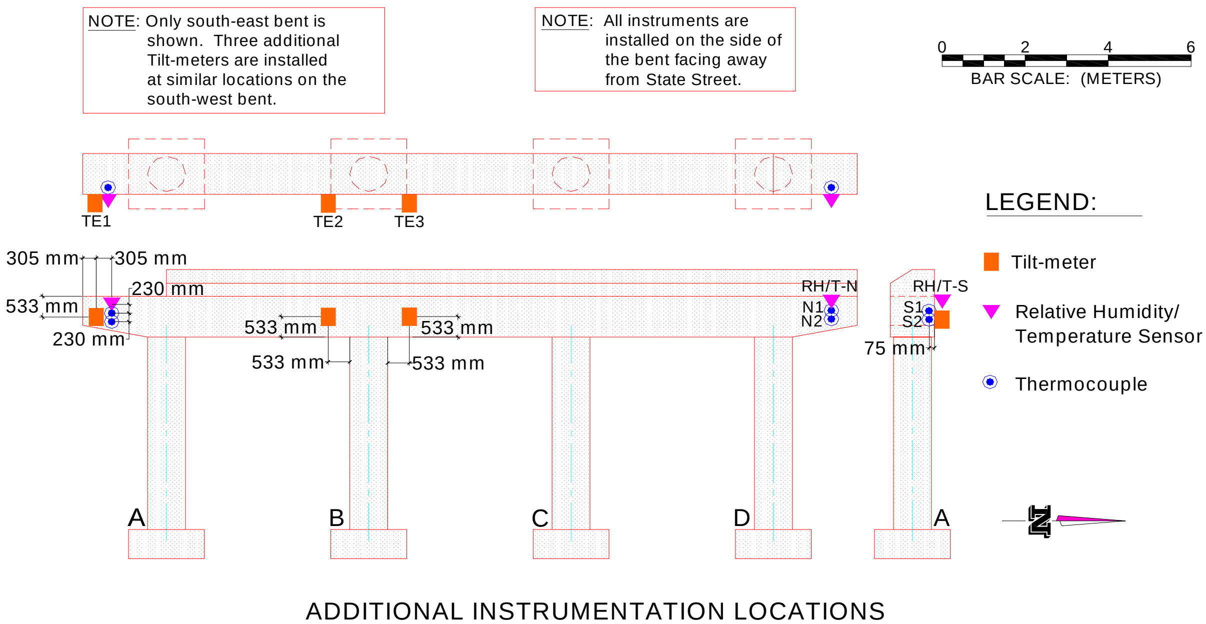


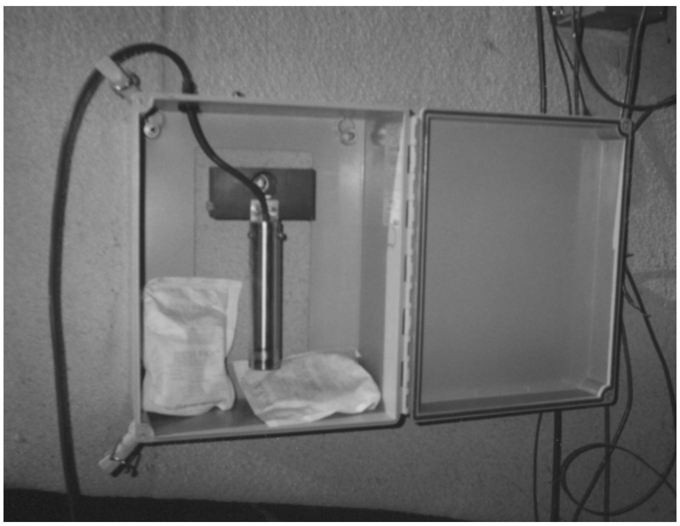
Figure 12.8. Sensor locations for the southeast bridge bent

(a) (b)

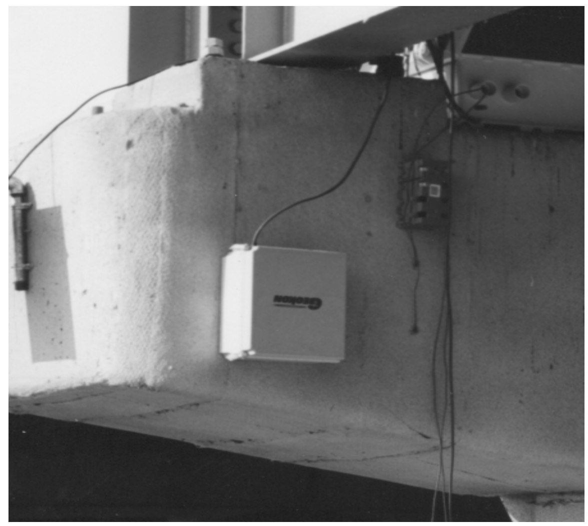
Figure 12.9. Location of thermocouples and relative humidity/temperature sensors on southeast
bridge bent: (a) south end; (b) north end

Tilt Meters

Another variable that was desired for the analysis of the retrofit was the deflection of the bent at various points, such as at the cantilever end and the midpoints between the columns of the cap beam. Since this is a field application open to the public, many of the traditional deflection measurement techniques were not possible. It was therefore decided that the best way to determine the deformation at these points would be to measure the rotation of the cap beam. Therefore, tilt meters were mounted at the critical points to measure beam cap rotation. Three tilt meters were mounted on the southeast bent and three in similar locations on the southwest bent. The locations of the tilt meters and the method of mounting are shown in Figures 12.8 and 12.10. The tilt meter was mounted to a bracket, which was anchored to the bridge bent. An enclosure was also anchored to the wall around the tilt meter in order to protect it and its movement.



(a)



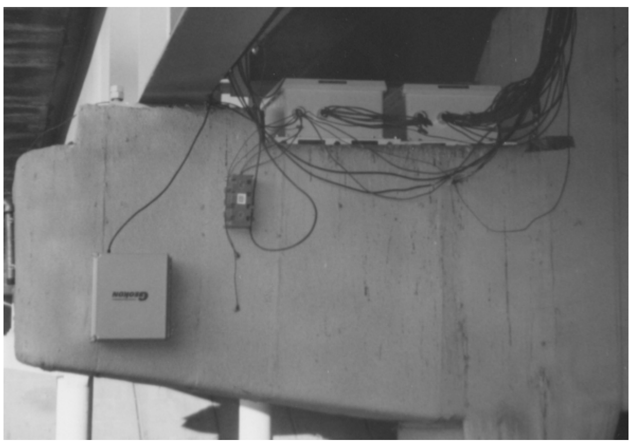
(b)

(c)

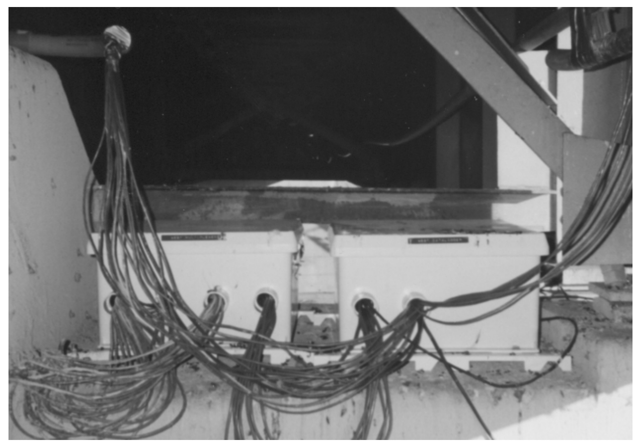
Figure 12.10. Location of tilt meters: (a) instrument; (b) southeast bent; (c) southwest bent

Data Acquisition System

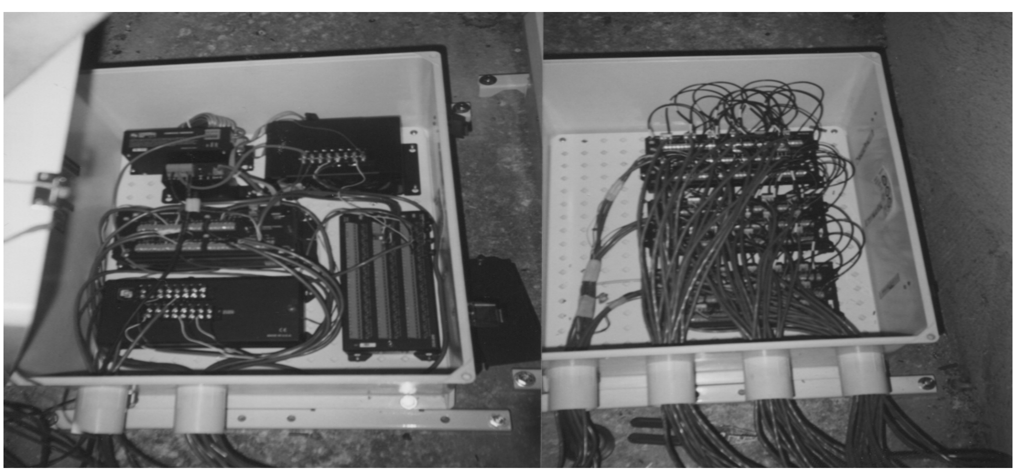
The data acquisition system, thermocouples, relative humidity/temperature sensors and tilt meters were installed on the bridge bents in January 2002. The data acquisition system includes a datalogger, multiplexers, power supply, and data retrieval equipment. These instruments are located in large plastic box enclosures. Two separate data acquisition systems were installed at the bridge: one on the southeast bent and one on the southwest bent; their locations are shown in Figure 12.11. Each sensor was connected to a multiplexer, which allows the datalogger to take readings from multiple sensors of the same type, through only one analog data channel. The dataloggers were then programmed to automatically take readings from each sensor every three minutes, and report the average reading every 15 minutes. This means that 96 data points for each sensor are recorded every day. A wireless Ethernet system was installed in May 2002 to retrieve the stored data from the dataloggers. This is accomplished by transmitting the data through an antenna to a receiver at the University of Utah, programmed to retrieve all newly stored data every three days. The dataloggers began recording data on January 19, 2002, and continue to do so at present.



(a)



(b)



(c)

Figure 12.11. Data acquisition systems: (a) southeast bent location; (b) southwest bent location; (c) inside enclosures

Thermographic Imaging

One of the greatest advantages to using CFRP composite as a structural retrofit for concrete structures is that it provides excellent confinement for the concrete, and therefore increases the load carrying capacity and overall ductility. Voids in the composite prevent the full tensile strength of the CFRP from being utilized, creating weak points in the confinement. The same is true for tensile strength application of CFRP, such as beam strengthening. It is therefore important to locate and eliminate any voids in the CFRP composite system.

For detection of localized voids or other bond flaws in field applications, coin tapping and thermographic imaging are generally used. It was determined that thermographic images would be most beneficial because they would not only locate voids in the composite, but they would also clearly define the size and shape of the voids.

In July 2003 several thermographic images were taken at various locations on the southeast bent of the State Street Bridge. Under natural conditions, voids do not show up in a thermographic image because the temperature is uniform throughout the CFRP composite application. Therefore, before the images were captured, selected areas were evenly heated using a heat gun. Areas that are properly bonded to the concrete are able to conduct heat into the concrete, as well as the surrounding composite and air. Any voids in the heated area would only be able to conduct heat into the surrounding composite and air. Because of the lower heat conductivity, voids retain more heat, and in a thermographic image, they show up as “hot spots.”

Several voids of varying size and shape were discovered within the CFRP composite retrofit system of the State Street Bridge. Figure 12.12 shows a thermographic image of a void discovered in one of the columns, which clearly shows up as a “hot spot” within the heated area of the column. Coin tapping verified that it was indeed a void. It was measured to be about 80 mm vertically, and 100 mm horizontally. Other smaller voids were also found in the beam cap.

It is unknown if these voids formed because of epoxy resin deterioration over time, or if they were the result of careless application of the CFRP composite to the bridge during retrofit construction. Continued monitoring of the voids is necessary to determine why the voids are present. If they are the result of epoxy resin deterioration, the voids will become larger with time, and possibly damage the surrounding CFRP composite. If the epoxy is not deteriorating, the voids should remain the same size and shape they are today. Under normal circumstances, it is recommended that any discovered voids be filled with epoxy resin to regain the tensile strength of the FRP composite.

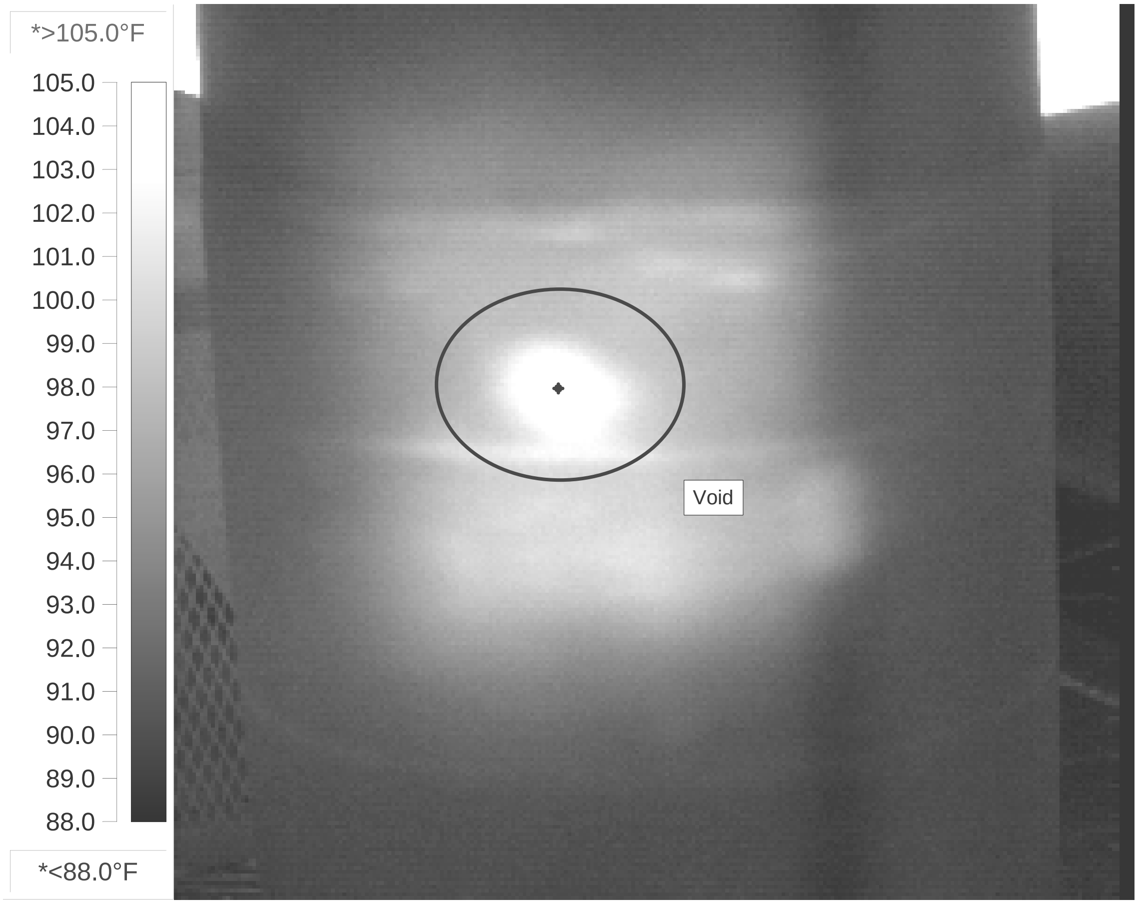


Figure 12.12. Thermal image of void in column

13. CONCLUSIONS

The seismic retrofit of State Street Bridge has been presented using a CFRP composite design. The design was implemented successfully in the summer of 2000 and 2001 while the bridge was in service. The CFRP composite seismic retrofit offers the advantage of being the fastest to complete and causes minimum disruption. The elements of the design included the columns, bent cap and joints. In addition to the CFRP composite seismic retrofit, other retrofit measures were implemented such as providing bumper brackets to engage the abutments and making the deck continuous over the expansion joints.

An evaluation of the as-built and retrofitted bridge bents was carried out for three design earthquakes. The as-built and retrofitted bridges were expected to survive the 0.2 g event. The as-built bent could be assumed functional after the 10% in 50 years event, but would probably sustain damage and might have to be replaced. The retrofitted bent is expected to survive the 10% in 50 years event with minor damage. The as-built bent would not survive the 10% in 250 years event but the retrofitted bent with CFRP composites is expected to remain functional.

Recent in-situ tests of RC bridge bents retrofitted with CFRP composite jackets under simulated seismic loads showed that the bents performed in a satisfactory manner (Pantelides et al. 2001, 2002b). The design of the CFRP composite seismic retrofit for the bridge bents tested was similar to the one presented in this article. This study demonstrates that seismic retrofit of RC bridges with typical pre-1960 design details is feasible using CFRP composite jackets.

The special provisions developed for the seismic rehabilitation of a reinforced concrete bridge were described, which included specifications, construction and installation requirements, and quality control and quality assurance guidelines. Several requirements of the specifications were implemented for the first time in highway bridge rehabilitation. These include: (1) utilization of "Strength Capacity" as the structural strength requirement for FRP jackets; (2) use of an environmental durability strength reduction factor to account for degradation of the CFRP composite; (3) fabrication of NOL rings during bridge site construction; (4) cooperation of the contractor with the owner in the performance of long-term health monitoring of the CFRP composite, during the CFRP composite application and while the bridge was in service; (5) requirement that the contractor procure the services of a manufacturer's representative of the CFRP composite material; (6) thickness gage measurement for inspection of the finish coat; and (7) a highly organized means of carrying out the sample preparation, collection, storage, identification, and testing of the CFRP composite test samples. The seismic retrofit of the I-80 State Street Bridge, and the strengthening of the columns of four other bridges with CFRP composites, using the supplemental specifications, were implemented successfully and involved epoxy injection of voids as the only remedial action.

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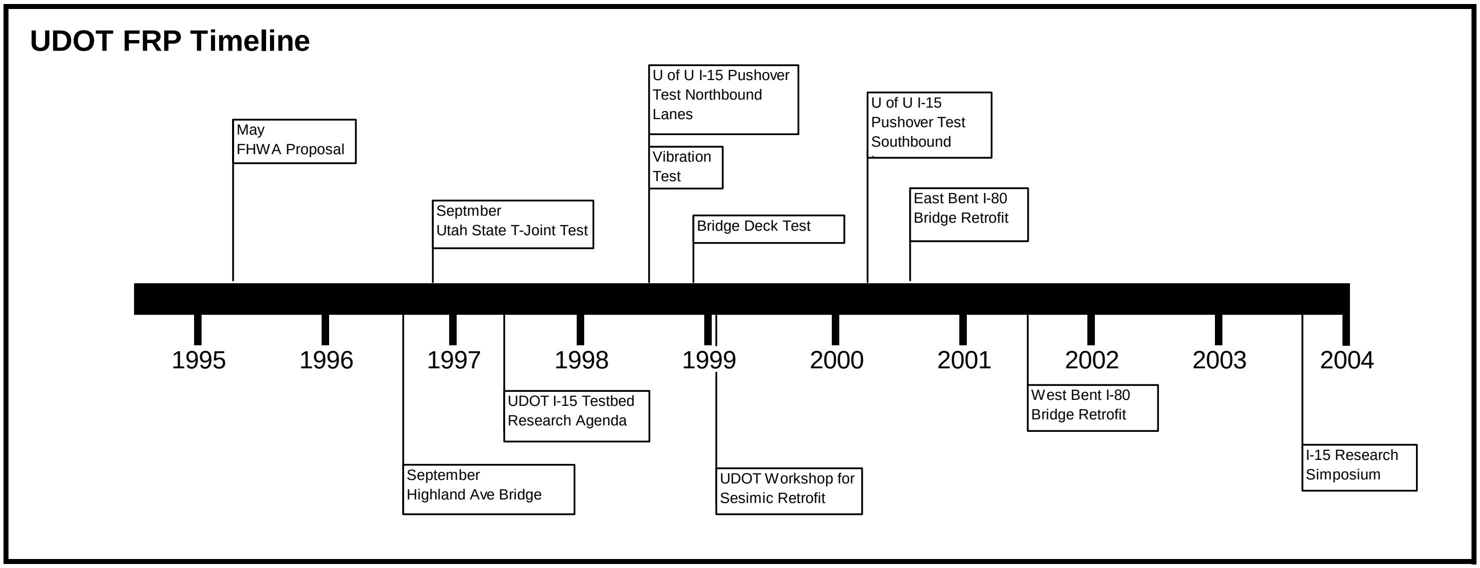

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